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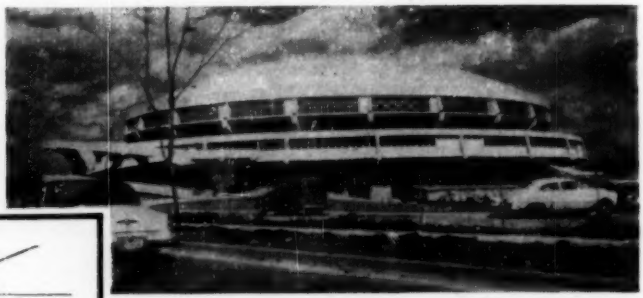
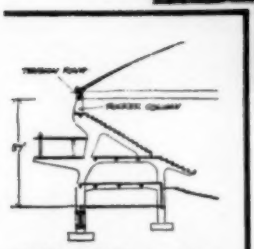
ACI
PROCEEDINGS
VOL. 55

# Championship Material





CROSS SECTION shows how monolithic dome and tension ring rest on hinged rocker columns, supported by stadium wall. Outer foundation is a 10' deep continuous ring girder.

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To absorb the stresses inherent in the flat shape of the dome (which rises only 40', or a 1:7 height-to-diameter ratio), and to withstand winds of hurricane force, the dome's perimeter connects with a prestressed concrete ring 3' thick and 5' high. This tension ring rests in turn on 24 reinforced concrete rocker columns, hinged top and bottom to absorb both internal and external stresses.

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JANUARY 1959

No. 7

Vol. 30

PROCEEDINGS VOL. 55

# JOURNAL
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"Tremie Concrete Controlled With Admixtures" prepared by J. WAYMAN WILLIAMS, JR., reveals results of intensive laboratory program conducted on tremie concrete.

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Two engineering educators, TUNG AU and THOMAS D. Y. FOK discuss the "Effects of Longitudinal Forces on Portal Frame Supporting a Highway Bridge Deck."

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W. C. HANSEN sets forth the findings of an investigation undertaken to explain the expansion and cracking that occurs in concrete made with certain aggregates in his paper "Expansion and Cracking Studied in Relation to Aggregate and the Magnesia and Alkali Content of Cement.

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"Wear-Resistant Concrete Construction" submitted by SHU-T'IEN LI presents requirements for materials, methods, and workmanship in constructing or resurfacing floors, platforms, and aprons subjected to heavy traffic and severe use.

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"Specific Surface of Aggregates Applied to Mix Proportioning" by B. G. SINGH suggests proportioning method, applicable to both continuous and gap-graded aggregates, using specific surface of aggregate as an index of grading.

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Title No. 55-48

# Concrete Space Structures—

# Relation Between Form and Structural Design

**Concrete has brought to the architect and structural engineer new techniques of design with fresh possibilities of form and shape. By its use, enclosed space can take many shapes other than the rectangle to which low tensile materials such as natural stone had once limited it. The evolution and potential of these concrete space structures is briefly presented.**

A. M. HAAS

● THIS SUBJECT OF SPACE STRUCTURES is born out of urgency. There is a great need for it. There exists a lack of understanding between the masters of form, and of structural design: the architects and the structural engineers.

Especially in the domain of concrete space structures are new techniques being worked out, new structures created, and fresh possibilities as to form and shape grasped. In the midst of a rapid and fantastic development of concrete construction, the architect can hardly keep up. Is this because the pace is too rapid and dynamic, or because architects have not learned to think in terms of three dimensional structures, or have not educated themselves to do so?

In the past, architectural physical planning was always limited, among other things, by the structural capabilities of the materials used. The stability of low tensile materials (such as natural stone) required huge, vertical, supports close together and aligned more or less on rectangular patterns. With the discovery of the possibility of vaulting for low tensile materials and the availability of concrete that can easily take the required form, supports were pushed much farther apart, and the rectangular ground plan could be abandoned. Enclosed space started to take other forms. Varieties of space

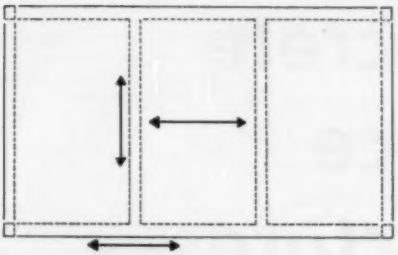

Fig. 1—Load distribution in post and lintel construction

arrangements and shapes were possible as a result of mass construction in brick, natural stone, and concrete. This gave Byzantine, Islamitic, Gothic, and Renaissance architects inspiration to manipulate space into forms unknown before. The recent introduction of materials strong in tension, i.e., steel and reinforced concrete, has further promoted this tendency in space manipulation. Supports were pushed still farther apart, with units of space taking more horizontal proportions.

Apart from a few striking examples, reinforced concrete has been used in the traditional forms of the older materials of construction. In timber as well as in steel structures, one-dimensional elements are used to build up the structural skeleton. Following this pattern, reinforced concrete structural members of the post and lintel (or beam) type were used. The sequence of load bearing, and consequently of the load distribution to the supports, may be described as panel (spanning in one direction) to beam, beam to girder, and girder to column (Fig. 1). Although easy to calculate, this in no way does justice to the special properties of reinforced concrete. It is not economical and in principle is noncontinuous and nonmonolithic.

## THREE-DIMENSIONAL STRUCTURES

To quote, such construction is "as economical as if three men carrying a piano would perform their task climbing on each other's shoulders with the top man actually holding up the piano" (Masur, Illinois Institute of Technology).

Now concrete may at any point take compression in every direction. We may define this in three dimensions mutually perpendicular to each other. Thus mass concrete may, at the same time, function in compression in three dimensions. If we confine ourselves to two-dimensional elements such as slabs, we then, for practical purposes, restrict this statement to two dimensions.

Reinforcement may be arranged in such a way that it takes tensions following a two- or three-dimensional scheme by using wire mesh or cube reinforcement. There are structures in which advantage is taken of this two- or three-dimensional distribution of stresses. To them belong the rectangular plates utilized in two dimensions, and also the circular plates. More typical,

and utilized in three dimensions are the flat slabs, shells, and reservoirs; they will be defined as concrete space structures.

A plate may be used for bending as well as for resisting forces in its plane such as axial load and shear forces, and thus create forms of maximum structural efficiency. The slab is an active structural element, monolithic in character, which should be designed, calculated, and reinforced on that basis. This is particularly true for the circular plate, and for the circular footing as well. Circular plates are more economical as to amount of material used. Designers too often fall back to the rectangular grid of beams for two reasons. They have been trained in the orthogonal scheme of thinking and consider circular plates more difficult to calculate because the formulas are not at hand.

A. M. Haas, professor of concrete and concrete construction, Institute of Technology, Delft, Netherlands, has many years experience in the field. His construction experience spans three continents. In addition to the Netherlands, he has worked in the United States, and the Netherlands East Indies. A member of ACI, Professor Haas has been on the faculty at Delft since 1953 and is also president of the Netherlands Concrete Association and of the Netherlands Committee for Concrete Research.

If a slab is curved out of its plane, we enter into structures of three-dimensional stature. Representatives are flat slabs and shells. Flat slabs are really three-dimensional structures (the columns belong to them) and capable of carrying heavy loads. Shells, on the other hand, excell in having little dead weight and are especially adapted to roof structures.

**Planning of space structures**

When a design is to be made, and this applies especially to the initial and preliminary design, all the requirements should be put together and to a certain extent each should be met. There are first of all the requirements of the space enclosure and the layout, the general plan. Then there are the demands as to lighting, thermal insulation, acoustics, ventilation, and air conditioning, also the adaptation of the building to the surroundings. Last, but not least, the structure itself should be so designed that stability, stiffness, and strength result. Finally the design should be an economical solution.

In the case of concrete space structures the last two items, structural requirements and economy, are of primary importance. We have developed such calculation techniques that almost any fantastic architectural design may be made structurally possible. These acrobatics, and the subservience of the clever structural engineer, certainly have disadvantages. The possibilities of analysis are such that the consulting engineer hardly calls a halt to the conceptions or caprices of the architect. If architect and engineer were cooperating on a more equal basis, and understood each other better, the result would be a more harmonious and well balanced design, and probably a more economical one.

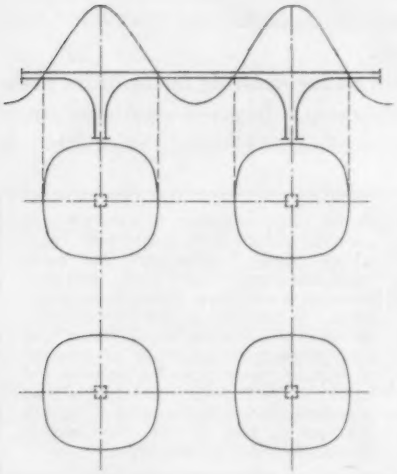

Fig. 2—Relative areas of positive and negative moments in flat slabs

## FLAT SLABS

Flat slabs, being three-dimensional frameworks, should be designed with a sufficiently large flare-head (column capital) that the negative bending moments over the column head may be taken easily and at low cost. The floor area in which these negative moments appear is comparatively small in relation to the large area taken by the positive moments (Fig 2).

The column capital usually slopes about 45 deg., allowing for a drop panel if needed. Formwork is rather practical and comparatively inexpensive, but provision for negative moments may lead to heavy and clumsy looking capitals, especially when large columns are needed. For warehouses and storage buildings, this does not necessarily look ugly, but indicates the bearing function (Fig. 3). However for all those structures not designed for heavy floor loads other forms, even a curved outline for the column capital, will serve as a better

Fig. 3—Typical use of column capitals in storage buildings

solution (Fig. 4). If the columns are cast first including the capitals, it allows for many re-uses of the formwork.

There is an increasing demand from architect's offices to do away with capitals. This is possible but expensive. For larger spans, provisions should be made to take the negative moments at the column head by inserting a steel structure at that location. This is an expensive means and should only be used in cases of necessity. It is much better to convince the architect that a capital—be it small—will be the right thing to have. Many floors having beams protruding below the slabs are everywhere accepted; therefore a small flare-head at the springing of the column should not be detrimental. In the Netherlands for housing projects, the column capital is already accepted and has acquired its place in architectural design.

**Uses of flat slabs**

Among progressive architects there is a growing tendency to use flat slabs on a column network of large spacing. In housing you will thus have only a few columns per flat. Furthermore it affords the architect freedom in placing partitions as he sees fit. The bearing capacity of flat slabs is such that they can always support a partition (of steel, brick, lightweight concrete). Moreover in multiple story construction the contractor has a series of continuous flat floors under which the remainder of building activities may be accomplished regardless of weather.

In industrial building too, little attention has been given to the specific character of the flat slab. Supporting columns are placed on the end lines

Fig. 4—Example of a more decorative column capital

Fig. 5—Full columns and column capitals at end bays aid in taking full advantage of flat slab construction

and column capitals are consequently cut in half. End bays similar to frames in traditional girder design are thus created. To build economically, a full-fledged column, including the capital, should be used for the end bays, requiring a setback of the first row of columns, with the edge of the slab at the approximate location of the zero moments (Fig. 5). This arrangement fits in well with the modern ideas of building construction in which there is a splitting of functions into two main items: the load bearing structure and the protecting skin (or envelope).

Attention should also be given to openings in flat slabs. It is well known that they should be made in the center part of a panel. If they are made in or near a column capital, large disturbances in the load result, and stress concentration may occur. In several cases a project clearly suitable for flat slabs could not be realized due to large openings (for staircases, elevators, etc.) near the columns. It is my experience that these adversities may be avoided by consultation between architect and designer at an early stage. It is of little use to redesign a given structure on a flat slab basis just before it is to be let for bids; the floor plan has been made firm and shifting of openings will no longer be considered possible.

## SHELLS

Shells are even more typical space structures. Here there are few restrictions as to shape. Any form of roof structure may be considered. There may be a close relation, or no relation at all, between architectural form and structural design. In the first case a close cooperation should exist, especially in those cases where we move toward the limit of what will be structurally possible. Then the designer will advise the architect what is still possible, and, for a given network of columns, dictate concrete dimensions and height. The second case may result in a misfit that is an unsuccessful solution. The

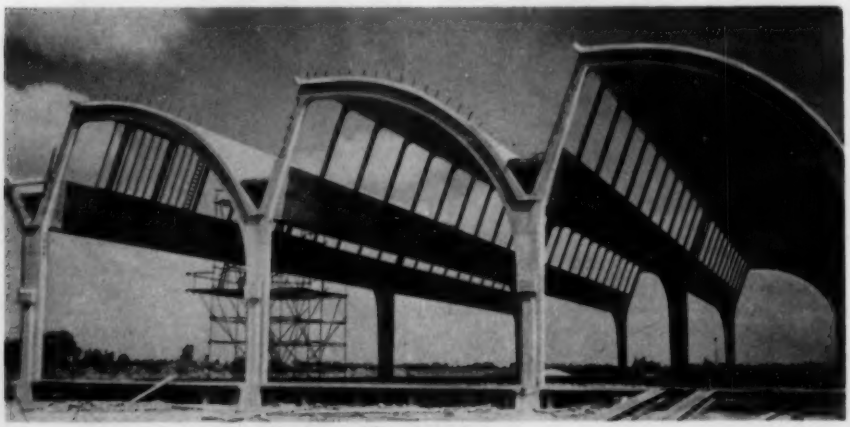

Fig. 6—Structural space frame for factory building

Fig. 7—Shell roof without edge beams

architect, no longer being the creator of form, may give little attention to the appearance as that of a space enclosing structure. He will just provide a skin or curtain around it and not care, or not know, about its functional behavior. A good example may be found in a factory designed by the author, a 22-bay north light structure. This had to be prestressed, and designed many times to meet the requirements of lighting, thermal insulation, and acoustics. It is a structure on the limit of possibility (131-ft spans). The architect did not touch the structural outline (Fig. 6). All he did afterward was to put a wall around it in which one circular opening per bay was used. He completely concealed the structure itself.

Attention may be called to the problem of how a shell should be presented in its outward appearance. The curved edge should be shown, or at any rate be seen; the question is whether this edge should be accentuated or not. This is a matter of circumstance and environment. The two extremes may be observed in the examples shown. The first one presents a number of circular shells built in Yugoslavia in which the edge is shown as an extension of the shell proper having the same thickness. In this case, with the shell projecting beyond the columns and designed without edge beams, there results a well chosen solution harmonizing with the surroundings: a shell without pretension (Fig. 7). To accentuate the curved circumference of a shell, the edges are often thickened. An example is a butterfly shell at Rotterdam Central Station (Fig. 8). There is a growing tendency to omit edge beams in

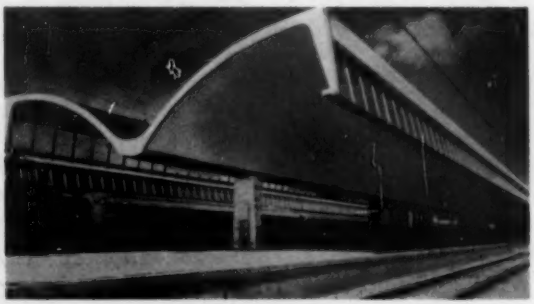

Fig. 8—Edge beams accent shell roof of railroad station at Rotterdam

the case of circular cylindrical shell vaults. This facilitates execution and at the same time enhances the beauty of the shell, giving it a loftier and more slender appearance.

**Double curved shells**

Double curved shells offer greater opportunities to designers and architects alike. These are the domes and cupolas appearing in gracious and classic forms. Recently the hyperboloids and conoids have come to the foreground. The latter are shells having two opposite curvatures, one in a positive and one in a negative sense.

Domes and cupolas are easily erected on a circular or elliptical ground plan. They more or less belong to each other. However it is by no means necessary to have a continuous solid curved edge beam to act as the lower and supporting rim of such a shell. The architect may also choose to support his dome at a number of individual (and isolated) points. A well known example is the MIT auditorium. When the architect decides to follow this course it is the plea of the designer that he choose points of support at regular distances on the circumference. This will make the designer's work, and especially the part done by the mathematician, much easier. A rectangular ground plan may even be used. There are a number of solutions, from having shells of transition in between supports to starting out with inclined arches of support, of which Nervi's Salone at Turin is an example.

Parabolic hyperboloids are not bound by any specific ground plan; they offer the architect more freedom to express himself in forms which his imagination may shape. These shells do have a few advantages, for one, the surfaces are made up of systems of straight lines which allow the use of straight reinforcing bars.

The height of the structure is not laid down in a simple rule. It may, to a certain extent, be chosen at random; the limit being a height of zero, e.g., the plane or straight plate. Here we find a problem often overlooked. The surfaces may be conceived as coming into existence by twisting a plane plate. This means that shear forces will be acting in the plane which also may be instrumental in transmitting axial loads. However shearing stresses always give rise to two kinds of principal stresses, one being tension and the other, perpendicular to it, being compression. Therefore, in contrast with the double curved domes, this type of shell will always have tension stresses to be resisted. As a matter of fact these two kinds of stresses may be observed and traced on the surface of the shell: parabolas of tension and of compression. The tension and compression forces become higher as the pitch of the shell becomes less.

**Prestressing shells**

The tensions may be of such magnitude that they cannot be efficiently taken by reinforcement, which has led to the application of prestress in a few

**Fig. 9—Folded plate roof in Denver shows architectural possibilities of this type of construction**

necessary cases. In the *Schwarzwaldhalle*, a better shell structure (from the designer's point of view) could have been achieved if the curvatures had been more pronounced. In both directions the shell has been prestressed; in the longitudinal direction 2000 tons of prestress had to be applied.

In several instances the shape of the shell is such that a classic calculation will not be possible or will not lead to reliable results. In that case two possibilities remain, one theoretical and one experimental. The shell may be assumed to be a membrane or to act as one. The bending moments emanating from the edges may be considered as disturbances of the membrane stresses and be calculated on that basis, at least in the case of axially symmetrical shells (having an axis of rotation). Another solution is to make a model on reduced scale, test it and take stress and deflection readings.

## FOLDED PLATES

Folded plates may be mentioned in this connection. They are sometimes used instead of shells for two reasons: they are easier to calculate, and they may afford savings in construction. There are several simplified methods for calculating these structures. They may be assumed to be a series of planes stiffly connected to each other and capable of transmitting large axial forces. Compared to shells we find that for equal spans the amount of concrete will be greater, roughly 1½ to 2 times, so that they will be limited to shorter spans. Interesting and striking architectural results may be achieved; as illustrated by examples from Denver (Fig. 9).

## PRECAST CONSTRUCTION

The architecture of precast structures is a chapter by itself, which will only be touched on. In considering precast concrete space structures we should reflect first on the underlying principles. In all cases we have to deal with structures built by assembling precast units, forged into a single unit by making the joints work together or by prestressing the whole, and the design should be executed accordingly.

In precast shells a new era is dawning. Nervi, in this respect a forerunner and master, has graciously combined precast and cast in place concrete. In

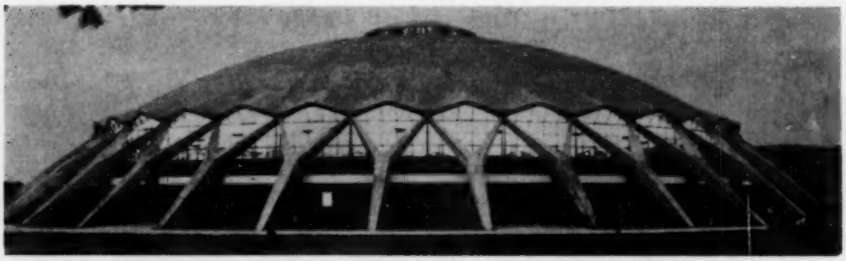

**Fig. 10—Sports Palace at Rome, Italy, by Nervi, is example of a shell dome made of precast units.**

his Rome sport palace (Fig. 10) he constructs a dome of 200 ft diameter by means of small precast diamond shaped pieces and then joins them together with a thin concrete skin on top.

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Title No. 55-49

# Criteria for modern specifications and control

EDWARD A. ABDUN-NUR

LEWIS H. TUTHILL

A PHILOSOPHY OF SPECIFICATION WRITING is advocated which will take full advantage of engineering knowledge, modern equipment and methods, and the statistical control of the end product. To accomplish this, "design criteria" for specifications, modeled after structural design procedures, are suggested.

An example of design criteria making use of "self-functioning" features, automation, contractor motivations, end-product provisions, use of reference specifications, uniformity requirements, investigation of local conditions and practices, use of statistical methods, use of labor reducing features, elimination of expressions such as "approval of the Engineer" which cannot be evaluated by the bidder, and in general, deciding what is needed and requiring it clearly, is given.

Realistic up-to-date specifications, trained and capable inspectors, and intelligent supervision by field engineers will result in maximum return for construction funds.

THE increasingly accelerated pace of construction, winter operations, shortage of qualified technical personnel, shortage of materials, rising costs, higher and more complex design standards, and the requirement that uniform quality be maintained on a project to increase the life of structures and decrease maintenance, necessitate the use of quality control methods in construction.

Quality control is the establishment of a proper level of quality required for a given project, and the controlling and conducting of all operations to provide a degree of uniformity that will assure, within a given degree of probability, that the required level will be attained. It will, by eliminating waste, save materials and lower costs. Uniformity is attained most readily by increased automation, which in turn lowers contractor's costs, as labor is usually the most expensive factor in construction.

Quality control cannot be attained, however, without proper specifications. These constitute, in fact, the first step in control. Unfortunately, most specifications are copied from others, originally written for entirely different situations. In addition, once a provision is written, it becomes static by the very fact it is in print in a relatively permanent document. This tends to

make specifications rigid and reduces the flexibility needed for proper quality control. Yet, specifications are in many ways as important as the design of the project.

Nevertheless, too often specification work is delegated to an employee of minor rank compared to that of the designer. Specifications should be as carefully "designed" as structural features of a project, and should be carefully tailored to each situation. They should, therefore, be entrusted only to

---

● EDWARD A. ABDUN-NUR, consulting engineer, Denver, served as a consultant on the Northern Illinois Toll Highway where he conducted a materials survey, set up quality control, and developed new standard specifications. Active in ACI since 1948, Mr. Abdun-Nur is currently a member of ACI Committee 214, Evaluation of Results of Strength Tests of Concrete; Committee 611, Inspection of Concrete; and Committee 617, Specifications and Recommended Practice for Concrete Pavements and Bases. Prior to establishing a private consulting practice, Mr. Abdun-Nur was affiliated with the U. S. Bureau of Reclamation; the Denver District, Corps of Engineers; and a number of other governmental agencies.

● LEWIS H. TUTHILL, concrete engineer, Division of Design and Construction, California State Department of Water Resources, Sacramento, has been an active ACI member since 1926. He is presently a member of the Board of Direction, and is chairman of ACI Committee 611, Inspection of Concrete, and ACI Committee 614, Recommended Practice in Measuring, Mixing, and Placing Concrete. Prior to 1956, he was chief of the Concrete Laboratory, U. S. Bureau of Reclamation, Denver, with which he was associated for 17 years. Throughout his professional career, Mr. Tuthill has been closely associated with specifications for concrete and with its control.

---

creative engineers with broad experience and with full understanding of design, construction, materials, contracts, control and inspection, and with a progressive philosophy of engineering. Creative specifications require men willing to use progressive ideas, willing to think a program through, and in the strength of their convictions, stand up and defend their work if need be.

There is no perfect specification or quality control, any more than there is a perfect design. This does not detract, however, from the value of incorporating in specifications and control the best thinking and the most applicable and progressive methods that will permit the job to be done satisfactorily and most expeditiously, and be of the desired quality. This will result in a structure that will serve the purpose for which it is intended with least delay and interference from maintenance, and which will have the longest possible economic life.

The close interdependence of quality control and specifications makes it unfortunate that too frequently specification and materials departments are separate entities. Ideally, the two should be merged or closely coordinated by one individual who, in addition to experience in these two fields, should also be well versed in design and construction. This is difficult to achieve in large organizations where specialization is the order of the day. Engineering consultants, who not only bring in a rounded viewpoint, but are unfettered by organizational precedence, often provide the best solution.

## DESIGNING SPECIFICATIONS

In "designing" specifications one can borrow from the design field, and start with a set of "design criteria" developed specifically for a given specification. Such criteria modified from one project to another as conditions may require, become "specifications" for the specifications themselves. The following criteria were used by one of the authors in developing the standard specifications for the Northern Illinois Toll Highway:

1. Choose quality desired and be willing to pay for it, rather than leave quality to chance by depending on the lowest price. Within the level of quality desired, take the lowest price.
2. Require as much uniformity as possible at the quality level decided on.
3. Use "self-functioning" features that operate properly without constant attention, such as staying with a certain material or source once it is chosen for the job. This reduces variations resulting from accelerated construction.
4. Specify automatic features that eliminate or reduce reliance on the human element, such as electric moisture meters, or interlocking of sequences in batching plants.
5. Write in motivations to make the contractor want uniformity and strive for it instead of being against it, such as a strength requirement that permits him to save cement with increasing uniformity of operation.
6. Provide penalties for lack of uniformity to influence the contractor to cooperate in getting uniformity, such as a provision for extra cement if tests indicate low strengths.
7. Use end-product provisions that permit maximum ingenuity to be used by the contractor to attain desired results.
8. Include, by reference, standards and recommended practices developed by unbiased organizations such as ACI, AASHO, ASTM, and others, in lieu of test procedures detailed in the specifications. Standards are important items in all careful engineering, but should be modified or replaced by other methods if the job situation makes it advantageous to do so.
9. Examine carefully and critically recommendations of manufacturers and trade associations, and require supporting evidence before adopting.
10. Investigate thoroughly local conditions and problems before starting. There is no substitute for thorough study of the local facts and conditions in developing realistic specifications.
11. Tie specification provisions to designs and control program so that desired end result can be obtained.
12. Use statistical methods wherever possible. This substitutes statistical probabilities and averages for guesswork.
13. Require proved methods, but do not close the specifications to features that can be adapted from other engineering experience, or that have enough evidence to assure success.
14. Permit flexibility as long as uniformity is maintained.
15. Keep restrictions at a minimum, unless experience has shown that they are needed to improve or facilitiate the work.
16. Specify provisions that reduce labor, both to do the job and to engineer the work, such as winter protection by insulation and lump sum items.
17. Avoid the use of brand names or "or equal." Specify requirements and allow anyone that can meet them to contribute to the success of the project.
18. Eliminate as far as possible terms similar to "discretion of the Engineer," or "approval of the Engineer." Some study of what really is required will usually eliminate need for these costly phrases used by the lazy or uninformed specification writer.

19. Keep requirements high:

a. One can relax when an absolute need arises, but cannot strengthen weak specifications without increasing cost and getting into trouble.

b. Relaxing, when required, is then from a position of strength which gives advantageous bargaining power.

## APPLICATION TO ACTUAL SPECIFICATIONS

### Choice between materials of same class

Each material within a class, such as aggregate or cement, has a certain degree of variability. In addition, there are variations between individual sources or brands in a class or group. Therefore, the fewer such individual materials used within a class, the less variability and the more inherent uniformity. However, this principle has to be applied judiciously and with care, as it can become restrictive beyond gain. Unless there are reasons for eliminating a specific material within a class, one can avoid restrictiveness by leaving the choice to the contractor, but requiring that once the choice is made, he must use the same material throughout the job. This becomes a "self-functioning" feature.

### Lump sum items

Lump sum items reduce the amount of measurement work demanded of undermanned engineering crews, and therefore contribute to getting the work done properly by releasing engineers to concentrate their efforts on more important functions. This is a self-functioning feature.

### Aggregates

Aggregates comprise the largest single ingredient in concrete. The more uniform the aggregates, the more automatically their variations can be measured and compensated for, and the more readily uniformity can be attained in the mixed concrete. Unfortunately, coarse aggregate segregates and suffers particle breakdown every time it is handled. In most operations, coarse aggregate has to be handled several times before it is incorporated in the concrete mix, and therefore this segregation and breakdown become important factors.

One remedial measure is to narrow the range in which each of the sizes of coarse aggregate is produced, thus making the particles in each size range more nearly of the same dimension, and reducing segregation in handling. It is accepted practice to separate coarse aggregate for paving concrete into two size groups. This reduces segregation, but does not eliminate it to the point of providing a high degree of uniformity in finished concrete. A four-size separation would provide a desirable degree of uniformity in the concrete, but, practically, a two way split is about all that can be expected on most work using aggregate up to a maximum size of about 2 in.

A second measure that is frequently used to compensate for segregation and degradation is finish screening, with the cleanly screened aggregate going directly to the batching bins. This achieves a three-way improvement:

1. Remixing immediately before use, thus reducing the effect of previous segregation and increasing uniformity.
2. Sharpening the separation between sizes, again increasing uniformity.
3. Most important of all, it rejects the fine undersize—the result of degradation—which also increases uniformity. This fine undersize has a most disturbing influence on the uniformity of the concrete mix.

Savings from elimination of restrictive stockpiling will, in most cases, compensate for the cost of finish screening.

Fine aggregate, on the other hand, does not segregate badly. However, variation in moisture content introduces large variations in consistency of the final mix unless proper compensations are made. Here, the problem becomes that of measuring moisture content by some fast and simple method to permit compensating for it during batching. Electric moisture meters, installed in the weigh hoppers, give instantaneous moisture readings and record them, and are reliable, dependable, and reasonably priced. These readings are a valuable aid in adjusting the amount of mixing water to maintain a uniform consistency despite variations in water content of the sand.

The allowable range of variation in fineness modulus of fine aggregate should not be stated in absolute terms, but an occasional "out of limit" should be allowed. In most cases a requirement that 9 out of 10 fineness modulus tests have to be within the limits of variation of 0.20 from the average of the 10 is reasonable. Many operations can meet narrower variations.

### Miscellaneous

Certain quality requirements, to insure that concrete with the desired properties and durability will result, can be controlled at the time of batching and mixing. They are:

1. Maximum water-cement ratio
2. Maximum slump
3. Air content range
4. Maximum size aggregate

Some engineers would add a restriction on the minimum cement content. This last restriction is not really needed in a well controlled operation, since the other provisions effectively control the minimum cement content. It may give some a feeling of confidence to have it definitely set forth, but in such a case, it should be sufficiently low to permit the contractor to get the most benefit and savings from a well conducted and uniform operation. These quality requirements are found in most specifications, but are mentioned here to stress the fact that other measures for obtaining uniform strength and economy are useless unless these factors are carefully controlled.

### Concrete strength

Control of the strength of concrete introduces many problems, basically because strength is not known until 28 days after concrete has been placed in the structure, and too late to do much about if it should be low. Most specifications require a minimum strength which must be exceeded. Statistical

analysis indicates that this is a specification impossible to meet. The probabilities are high that no absolute strength requirement can be met 100 percent of the time. It is, therefore, more realistic to take a calculated risk based on probabilities arrived at by statistical methods, and allow a certain percentage of "lows." By permitting a reasonable number of lows, the engineer can hold the contractor to a standard that is possible of attainment, and therefore can be enforced. The following example will clarify this concept.

Assume that a paving concrete which develops a strength of 4000 psi at 28 days is required. Table 1 lists the requirements for average strength which will have the probability of attaining the strength required under the assumptions shown at the top of the respective columns and under various conditions of variability. These values have been rounded to the nearest 100 psi.

**TABLE 1—REQUIRED AVERAGE COMPRESSIVE STRENGTH AT 28 DAYS FOR 4000-PSI CONCRETE WITH THE PROBABILITY THAT THE CONDITIONS SHOWN AT TOP OF COLUMN WILL RESULT**

| Coefficient of variation, percent | Proportion below specified strength | | |
|---|---|---|---|
| | 1 in 5 | 1 in 10 | 1 in 200 |
| 5 | 4200 | 4300 | 4600 |
| 10 | 4400 | 4600 | 5400 |
| 15 | 4600 | 5000 | 6500 |
| 20 | 4800 | 5400 | 8300 |
| 25 | 5100 | 5900 | 11,200 |

The column "Coefficient of variation" provides a numerical measure of the variability of the control. The smaller the coefficient the better the uniformity. A coefficient of 5 percent is rarely obtained in field operations, while a coefficient of 25 percent is not uncommon. Operations range between these figures. These generalizations are based on probabilities for a whole operation, covering several months under uncontrollable seasonal and job variations, having a sufficiently large number of tests to constitute a representative and dependable sample of the total concrete placed.

The strengths in the column headed "1 in 200" indicate the unrealistic implications of specifying an absolute minimum, as even with an allowance of ½ percent of the test results below the required strength, and an assumption of good control (coefficient of variation of 15 percent), the average strength of 6500 psi required to be reached becomes out of reason. For poor control, the required average of 11,200 psi begins to sound astronomical. The number of lows can be reduced by raising the average, but cannot be completely eliminated. It is, therefore, good engineering practice to take a calculated risk and decide on a percentage below the design figure that can be tolerated without danger. For a structure like a pavement, where a failure results only in a crack that involves little risk to life, as many lows as 20 percent below design strength may be justified. The table shows that for reasonably

good control (coefficient of variation of 15 percent), the requirement is for an average strength of 4600 psi, which is not unreasonable. For poor control (coefficient of variation of 25 percent), on the other hand, the required strength goes up to 5100 psi which begins to show up in excessive cement content.

For a structure in which a failure has more serious consequences, a requirement restricting the lows to 10 percent may be in order, although there are engineering organizations that allow as high as 15 or 20 percent. This still provides a safe structure for the following reasons:

1. Experience has indicated that the strength of concrete in structures exceeds the strength shown by cylinders cured under standard conditions.
2. Factors of safety are used in design work to take care of such variations.
3. Individual batches are not usually concentrated at one point of a structure, but are mixed with other batches and distributed over a relatively large area, thus introducing an averaging process.
4. Surrounding concrete adds support and confinement.
5. Variation is often in testing and not in the structure.

#### Water-reducing agents

Water-reducing agents are of special interest for improving quality control based on uniformity. In the first place, by varying the amount of agent added to the mix, a uniform water and cement content can be maintained throughout the year in spite of temperature and weather variations. This, in itself, is an advantage in obtaining uniformity. In the second place, these admixtures provide higher strengths by permitting a lower water-cement ratio, or lower cement factors by reducing the water requirements and maintaining a given water-cement ratio. Such compounds also tend to reduce many of the difficulties that develop in hot weather concreting. They help by extending the time during which concrete can be revibrated, thus avoiding many cold joints; by retarding the set sufficiently to permit finishing with less difficulty; and by reducing hot weather water requirements.

#### Hot weather concreting

Many hot weather difficulties are traceable to higher temperatures of the mix. By specifying a maximum temperature of the concrete as placed, it becomes immaterial whether the contractor uses cold water to cool his mix, or ice, or cools his aggregates. The advantages through lower concrete placing temperatures, of lower water requirements, reduced shrinkage cracking, better quality and durability, higher ultimate strength, and lower range of contraction to the lower average temperatures the concrete reaches after hardening, are all well known.

#### Cold weather concreting

Cold weather concreting can also be specified in such a manner as to induce the contractor to choose methods that are most desirable, without placing undue restrictions, simply by making the desirable methods more profitable to him. Cool placing reduces loss of slump, avoids fast set, keeps water require-

ments low, and reduces the thermal shock to cold air temperatures. The trend is to place as close to 50 F as possible, in cold weather when mean daily temperatures fall below 40 F, and permit placing as low as 40 F in moderate weather. By the simple expedient of using insulating blankets or other insulating materials, the temperature of concrete can be kept from falling below 50 F through heat generated by hydration of the cement, thus resulting in low cost winter protection. Such protection is foolproof, free of the human element, and safe from dangers of fire and carbon dioxide. With the added use of 1 percent calcium chloride in the mix, and curing compounds on exposed surfaces or after form stripping, winter protection under these circumstances can be reduced to 72 hr, provided that care is taken to avoid thermal shock when removing the protection. Few, if any, contractors would choose artificial heating methods if given this choice.

#### Curing methods

Other conditions permitting, if the contractor can choose among the various methods, he is most likely to choose the pigmented curing compound over other more cumbersome and expensive methods. From the control standpoint, it is a self-functioning feature, since once applied, it needs no more attention. Available equipment applies curing compound to pavement surfaces automatically, and therefore provides advantages of uniformity and automation.

#### Truck mixing

Truck mixing cannot be controlled as well as or easily as central mix concrete or concrete batched and mixed at the placing end because it is impractical to send an inspector with every truck, and truck drivers may take liberties with the addition of water, the speed of mixing, and mixing time. One way to control this operation (until such time as equipment is developed to overcome the shortcomings), is to require addition of all water at the placing end, where an inspector can be in attendance.

#### Batch plants

Semiautomatic or automatic batch plants are almost obligatory on highway work as the speed with which batching has to be done for an efficient paving project is such that manual plants would delay the work. At peak operation of toll highway work, 120 to 150 batches per hr have to be batched to maintain full paving operations. The semiautomatic plant, where everything is automatic except dumping of the weighed batch into the truck, does the job effectively. Provided with recorders, and interlocks on the various gates, such a plant is nearly foolproof. Recorders, when provided, furnish a permanent record which supplements normal inspection.

All the features discussed above for controlling concrete operations help in attainment of the desired end product—a concrete with sufficient workability for placing, sufficient air for durability, and a minimum of cement needed to attain the required strength.

## INSPECTION

Language and requirements of specifications, no matter how complete and clear and forward looking, are only empty words unless they are carried out on the job by effective inspection. Some practices, like insulative protection of concrete in winter or sealing compound for curing, when once properly performed, are self-functioning in producing the desired results with practically no further inspection. But there are many other practices, like the placing and vibrating of concrete, which require continual vigilant batch-by-batch inspection if results of intended uniformly good quality are to be assured.

For this reason, along with the development of specifications, a continuing program of training and instructing field engineers and inspectors in the importance and meaning of specification requirements is essential. Only thus is teamwork created which transforms specifications from words into reality.

## FUTURE DEVELOPMENTS

As we look into the future, there is promise of several developments which will enhance considerably present means of improving uniformity, quality, and proper control of concrete making. Foremost among these are:

(1) Mechanical filters or other devices for reducing the moisture content of sand to a uniform and stable minimum.

(2) Equipment that will automatically adjust batch weights to compensate for variation it detects in the moisture content of aggregate.

(3) A method of beneficiating the particle shapes and surfaces of those sands and other aggregates which cause a high water requirement. With a range of from 220 to more than 300 lb of water required per cu yd in concretes that are similar except for aggregates, the benefits and economy from such development should be stimulating.

(4) Batching plant equipment arranged so that a new mix can be simply dialed by mix number and the plant will reset itself correctly, with proper adjustment for moisture in the aggregate.

## CONCLUSIONS

The elements of specifications recommended here have all been tried on various engineering projects and proved to be practical. They will assure the maximum of uniformity and therefore of quality control. With the type of control and specifications advocated here, prices come down, bidders are more anxious to bid because of the opportunity to use ingenuity and increase profits and because of the lowered costs brought about by automation. The owner is better satisfied because he gets a better product with a longer life and reduced maintenance, in most cases at lower first cost.

The type of quality control and specifications advocated above requires creative engineering with full understanding of design, construction, control,

materials, contracts, and a progressive philosophy of engineering. It requires continual training to produce a team of inspectors who transform the words of the specifications into reality. For these results, such work must be placed in the hands of broadly experienced engineers who can administer the work fairly, can meet problems with quick and equitable decisions, and who are interested in and sympathetic to the objectives of uniformity and quality in their rightful place, along with speed, production, contractural relations, and administration.

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Title No. 55-50

# setting time of concrete controlled by the use of admixtures

*By Raymond J. Schulz*

**TERMINOLOGY AND DEFINITIONS of setting time are discussed, factors influencing setting time are reviewed, and methods of controlling setting time are described. Examples of construction projects that have successfully utilized set control to improve concrete quality and solve special problems are given.**

WHAT IS SETTING TIME? Different trades define it differently. To some it means the time when concrete loses slump or workability, to others when concrete floors can be finished, when cold joints would occur between successive lifts, when pressure on forms ceases, or even when forms can safely be stripped.

All these definitions of setting time are important to the contractor, engineer, or concrete supplier. For example, concrete placed in a conventional form may be mixed at a central mix plant and transported to the job site. Until it is placed there should be no loss in workability. The time it takes to stiffen is, therefore, the "setting time" that the contractor is interested in. After placing, the concrete must not *set* before it is covered by the next layer, or cold joints will form. The loss in workability is not a factor here, but the "setting time" means that the concrete is alive enough to bond completely. Forms must be designed to withstand the hydrostatic pressure of the plastic concrete until it is *set*. This is the basis for another definition of setting time.

Another type of "setting time" which is encountered is in concrete slabs or floor work, and knowing when to finish the slab can help the contractor in his operations and insure a good finish for the owner. In the case of bridge decks, such as composite bridges or continuous girder bridges, the setting time will determine the size of panels which a contractor can place. In this type of construction, the forms or girders will deflect due to the weight of the freshly placed concrete. If deflection occurs when the concrete is no longer plastic, cracks will develop.

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Setting time must be defined. Arbitrary points must be taken on the setting curve, and the continuous process of loss of workability and gain in strength must be correlated with the needs of the engineer in the field.

## FACTORS INFLUENCING SETTING TIME

Many factors affect setting time of concrete. In fact, even the definition of setting time is rather broad, uncertain, and arbitrary. Setting, even of neat cement paste, is a continuous process. When testing setting time, arbitrary points are determined on a curve. For portland cement, these points are easily determined in the laboratory by use of the Vicat or Gilmore needle. But such testing provides the engineer or contractor in the field with little information as to when his concrete is going to have the stiffness or strength he needs for specific purposes. In the laboratory, normal consistency is used with the water-cement ratio so low it would rarely, if ever, be used on a construction project. In the laboratory, temperature and humidity are always uniform, everything is standardized. But in the field, temperature and other conditions will vary.

The following factors will influence the setting time of concrete:

- Variations in the cement, type, brand, composition, and age
- Temperature of the mix
- Ambient temperature
- Cement content of the mix
- Size of the concrete member
- Slump and water-cement ratio
- Bleeding characteristics of the particular mix
- Admixture used

Where concrete is to be used in flat slab work, the following factors will also affect setting time:

- Relative humidity
- Wind velocity
- Radiant heat of the sun
- Absorption of the subgrade or forms

### Type of cement

There are five recognized types of cement, but only three available for general use, the normal Type I portland cement, the Type II or moderate heat of hydration cement, and Type III high-early-strength cement. Although ASTM specifications allow the same setting times for all three types of cements, in general, the high-early-strength cements set the fastest, followed by Type I and Type II. There is a much greater difference in the rate of strength development in these cements, with high-early-strength cement doing

exactly what its name implies, and Type II cement developing strength rather slowly. Different brands, or the same brand from different cement mills may also have a different setting time. As an example, the Lehigh Valley in Pennsylvania is only about 150 miles from the Hudson Valley in New York, yet most Hudson Valley cements set faster than the Lehigh Valley cements, even of the same brand name. This is due to the nature of the raw materials available. In regard to strength, durability, and all other factors, excellent cements are produced in both regions.

The cement manufacturer regulates the setting time of portland cement by the addition of gypsum so that it conforms to ASTM specifications, the test to be made under standardized conditions. These specifications are broad enough to permit portland cements to pass even if setting times vary greatly between types and brands.

The more finely ground a cement of a given composition is, the faster it will set. More surface area is exposed for reaction with the mixing water.

**Age of cement**

Cement may change its characteristics with age, particularly if it is allowed to slowly absorb moisture from the air as sometimes happens during storage. Cement fresh from the mill often shows flash-setting tendencies. If it is still hot, it will set faster on account of the resulting higher mix temperature. Generally, the older the cement, the slower it will set.

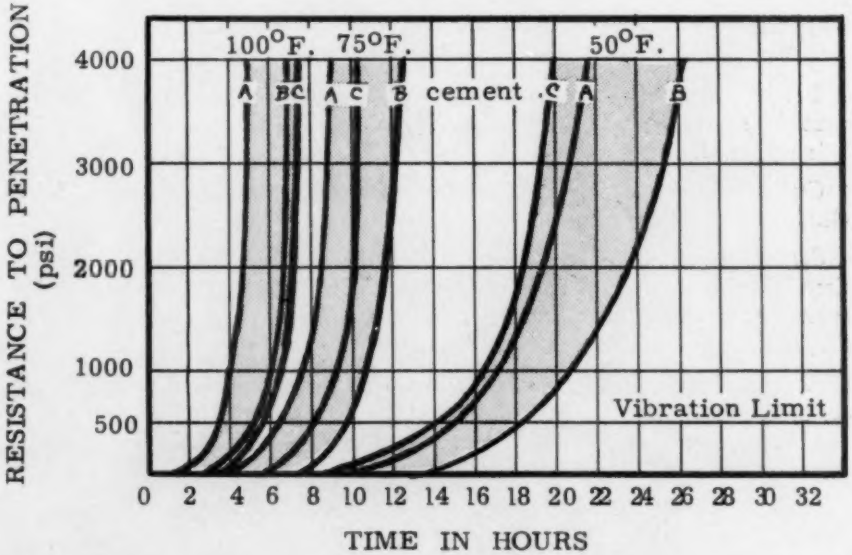


Fig. 1—Temperature and brand of cement influence hardening characteristics of concrete mortar

### Temperature

The temperature—both initial mix temperature and the ambient temperature—will have one of the largest effects on the setting time of concrete. When portland cement sets, a chemical and physical-chemical reaction takes place. Like all chemical reactions, the reaction of cement and water is accelerated by heat and retarded by cold. An old rule says that each 20 to 30 F increase in temperature doubles the rate of chemical reactions. The influence of temperature is therefore enormous; cement mixed at 90 F hydrates or sets four times as fast as cement mixed at 40 F, (see Fig. 1) as was reported by Tuthill and Cordon.[1]

The hydration of cement involves a physical-chemical phenomenon causing formation of the cement gel or initial reaction product. The rate of this gel formation is governed by the temperature, which means that temperature not only affects setting time, but, as temperature increases, the concrete will lose slump faster because more water is being used in the formation of larger cement gels (see Fig. 2) as reported by Price.[2]

### Physical characteristics of the concrete

Although the speed of a chemical reaction, such as hydration of cement, is not changed by greater or lesser dilutions, the apparent setting varies; [ (Fig. 3) as reported by Tuthill and Cordon[1]]. What is being measured and what interests the engineer is not whether hydration has started or to what degree it is underway, but what the apparent stiffening effect is. If, for instance, water is withdrawn after placement, using the vacuum process, concrete will "set" and have sufficient strength to support its own weight within less than 10 min after mixing, when hydration has hardly started.

The influences of the cement content and the other factors listed below are caused by the physical characteristics of the mix and the conditions to which it is subjected.

*Cement content*—A high cement content results in a rapid build-up of heat. Gradually increasing concrete temperatures cause faster set.

*Size of the member*—The size of the concrete member will influence setting time. In a large mass of concrete, the heat generated by the initial reaction cannot be dissipated very quickly, and the temperature of the concrete will rise, accelerating the chemical reaction.

*Slump*—Slump will also affect the setting time of concrete. High slump concrete will set more slowly than low slump concrete. The excess water added to lubricate high slump concrete will take longer to bleed and evaporate; therefore the mix will remain plastic longer.

*Bleeding characteristics*—The bleeding characteristics of a particular mixture will affect the hardening rate and setting time. A mixture which bleeds rapidly will lose the excess water added for workability at a fast rate and become stiff sooner.

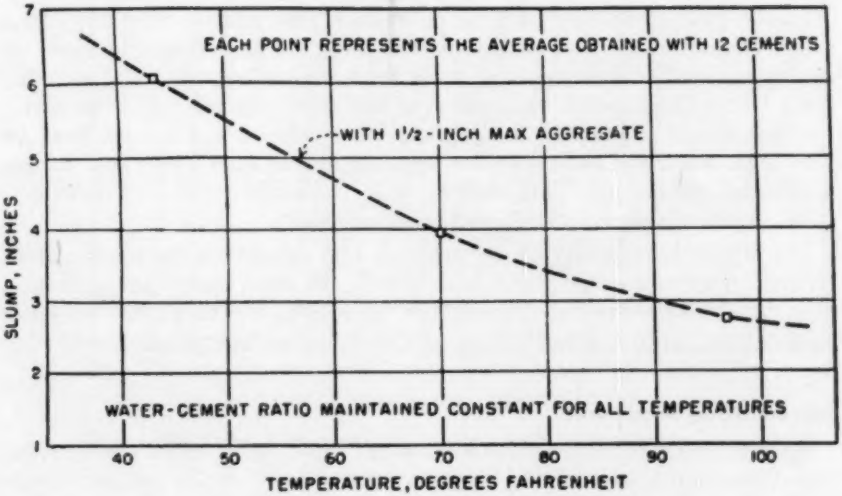


Fig. 2—Loss in slump due to rise in mix temperature

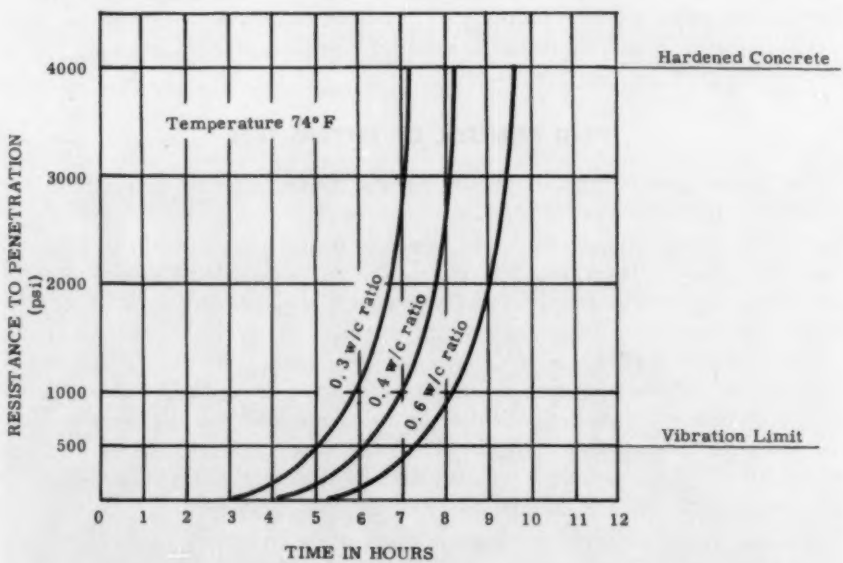


Fig. 3—Richer concrete mortars (lower water-cement ratio) harden more rapidly

### Other influences for slab and floor work

In the case of concrete slab or floor work, the relative humidity, wind velocity, and radiant heat of the sun will affect the hardening rate, since the mixing water will evaporate faster when wind and sun are strong and humidity low. These three factors may often be the most important in determining finishing time. If humidity is low, and wind velocity and radiant heat are very high, water will be evaporated from the slab at such a rate that the top of the slab will dry out. The concrete will "caseharden," be very difficult to finish, and probably exhibit plastic shrinkage cracks.

The absorptive capacity of the subgrade also determines the rate at which concrete dries and excess water is removed. All other factors being equal, a slab placed on nonabsorbent forms (steel pans, etc.) will take longer to set or harden than concrete placed on an absorbent fill such as cinders or crushed stone.

### Set-controlling admixtures

Set-controlling admixtures should only have this one effect on the concrete, since the quantity used will, by necessity, have to be varied to suit job or temperature conditions. As an example, an admixture composed of an accelerator and an air-entraining agent could be used only in one proportion—using a larger quantity to accelerate the set of the concrete further might increase the air content of the concrete beyond allowable limits. Further, a set controlling admixture, whether it be a retarder or an accelerator, must not have a deleterious effect on any property of concrete considered essential for that particular use. Sugar, for instance, is a very effective retarder, but may not be necessarily good for concrete, as such.

## FIELD CONTROL OF SETTING TIME

All factors which control the setting and hardening rate of concrete are additive. It can be realized that a complicated situation exists.

Due to transportation costs, the cements which are available locally will have to be used. By careful job control the water-cement ratio can be regulated within practical limits. The concrete mix proportions can be controlled either by the testing laboratory or the engineer in charge. The size of the concrete member, of course, is determined by structural necessity. It is often possible to control the mix temperature or the temperature surrounding freshly placed concrete in the winter by using insulation or artificial heat.[3,4] In summer, shading, cooling aggregate, or adding ice as part of the mixing water are effective measures. In the case of slab work, tarpaulins or other windbreaks can be used to control wind velocity.

By use of admixtures, the setting time of concrete can be varied from 15 seconds up to several weeks, and such setting time can be predicted. The use of admixtures for such purposes is a very valuable tool of construction (see Fig. 4).

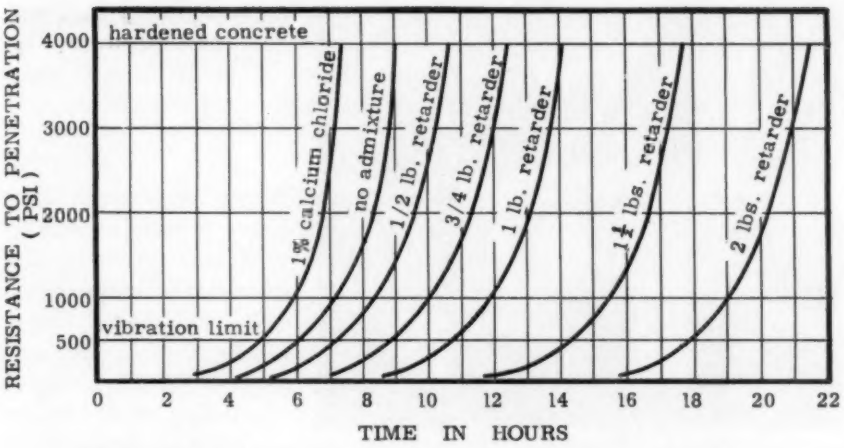


Fig. 4—Effect of various admixtures on the hardening rate of concrete

The control of setting time of concrete often solves many concrete field problems. Benefits obtained may be increases in quality, reduction in cost, the speeding of job progress, or a combination. Here are some examples of problems successfully solved with proper set control.

**Sealing pile cap forms**

The west approach of the Tappan Zee Bridge is constructed of abutments supported on clusters of timber piles (Fig. 5). The specifications required that all concrete for these pile caps be placed in the dry. The contrac-

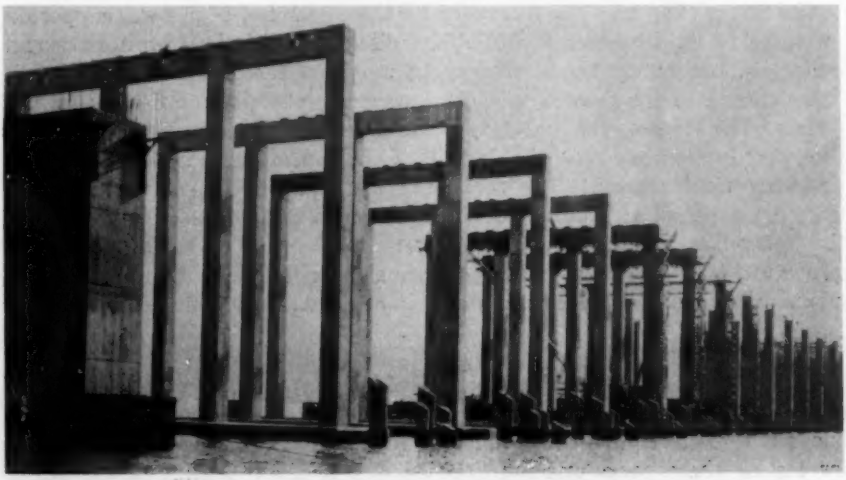

Fig. 5—Pile bents, Tappan Zee Bridge

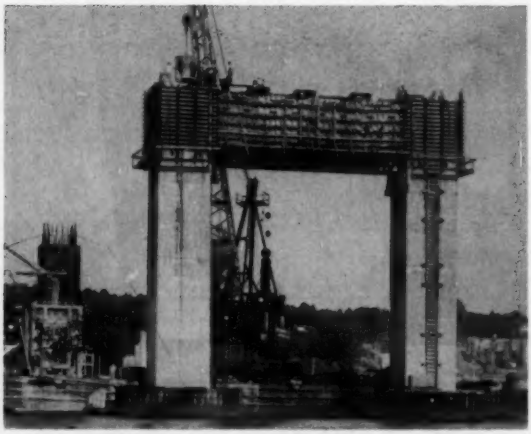

Fig. 6—Spandrel beam, Tappan Zee Bridge

tor was faced with the problem of having the bottom of the pile cap form 3 ft below mean low tide, and sealing formwork around the piles was quite a problem. The contractor solved it by constructing the forms 18 in. below the specified depth of the pile cap. By placing 18 in. of concrete, made very fast-setting by the use of an accelerator, the contractor was able to dewater the pile cap forms the same day the quick-setting concrete was placed. The quick-setting concrete formed a perfect seal for the concrete cap itself, which, as specified, was then placed in the dry.

#### Elimination of cracking due to form movement

On the same project, the contractor and engineer worked together to prevent cracking of the large 70 ft long spandrel beams used on the east approach to the bridge (Fig. 6). The formwork for these spandrel beams was supported on three 36-in. I-beams. Calculations disclosed that the formwork would deflect approximately 1½ in. due to the weight of the concrete to be placed. The capacity of the floating mix plant was such that it took 8 hr to fill the forms. The formwork would deflect continuously, and a part of this deflection would take place after the first lift of the concrete was no longer plastic. Cracking would result. The contractor retarded the set of the concrete, using 1 percent of an adipic acid type retarder in the bottom lifts and less in the upper lifts. Concreting was completed in 8 hr. The entire 14 ft deep girder was then consolidated by revibration using internal vibrators. In this manner the beam was placed and hardened as a monolithic unit with no cracks.

#### Elimination of cold joints

The center span of the Tappan Zee Bridge is of cantilever steel construction. Eighty percent of the dead weight of this span is supported by the buoyant force of eight hollow concrete boxes. The only available dry dock for casting these boxes was not deep enough to allow floating of the boxes if they were completed before launching. Casting of the boxes was therefore

completed to within 7½ ft of the top; then they were launched by flooding the natural dry dock. After the boxes were towed into the navigation channel of the Hudson River, concreting operations were resumed. However, the weight of the concrete to be placed was such that the draft of the boxes would increase and the bottom lift of newly placed concrete would be below the water line.

The probability existed that river water would infiltrate the forms and damage the unset concrete or possibly sink the boxes. Placing in conventional separate lifts would have resulted in undesirable construction joints and added expense and clean up. The problem was solved by placing the concrete in three 2½-ft lifts. The first lift contained ½ percent of an adipic acid type retarder. On completion of this lift a second lift of concrete 2½ ft deep was placed containing 2 percent of an adipic acid type retarder. The two lifts were revibrated together, eliminating a cold joint. The job was closed down and 18 hr later the third 2½-ft lift was placed. At this time, the first lift had set and gained sufficient strength to withstand the water pressure. The second lift was still plastic and could be revibrated with the third lift.

In this manner two construction joints were eliminated and there was no danger of water infiltrating freshly placed concrete. This schedule was carried out on all eight boxes.[5]

Setting time governs the optimum time at which concrete can be revibrated to effect an increase in strength and improve appearance. Fig. 7 illustrates that the slower the set, the later and more effectively concrete can be revibrated. It is also shown that the optimum time for revibration is not as critical if set is retarded.

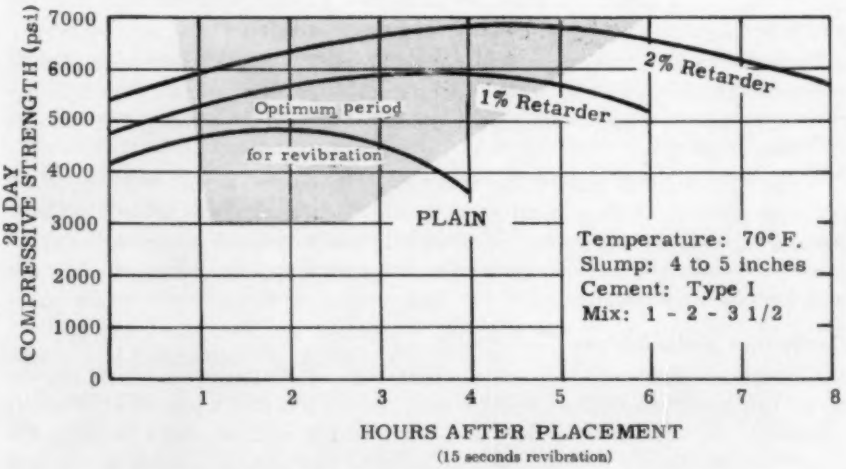


Fig. 7—Revibration of concrete

Fig. 8—United Electric Railroad Tunnel, Providence, R. I. Roof section before repairs (above) and after

**Sealing of leakage**

Repair work quite often requires control of the set of concrete or mortar in its most extreme form. The repair of a tunnel, owned by the United Electric Railroad, Providence, R. I., is a case in point (Fig. 8). It was carried out by sealing leaking areas with pneumatically applied mortar, flash set with an admixture. Hand sealing of leakage is time consuming and by using flash-setting pneumatically applied mortar, leakage was sealed at a very fast rate. Calcium chloride could not be used as an accelerator when sealing leakage because water easily washes through the mortar before it is completely hard. Further, to flash set shotcrete mortar with calcium chloride could develop so much heat that cracking might result.

Isolated cracks could not be sealed economically with pneumatically applied mortar. These were hand sealed using a quick setting mortar, produced by mixing portland cement with a good quick set admixture causing the mixture to set in 15-20 sec.

Voids and fissures behind the concrete arch of the tunnel were filled by pressure grouting with cement-sand grout. A retarder was added to delay the setting time of the grout. This added time permitted pumping the grout greater distances from one drill hole, insuring complete filling of all voids and fissures economically.

**Continuous girder bridges**

Continuous girder bridges are another type of structure in which control of set can mean the difference between a perfect job and a job which exhibits cracking. As the concrete deck is placed on continuous girder bridges, the weight of the freshly placed concrete deflects the girders, causing a negative moment which will crack concrete if it is no longer plastic. The normal

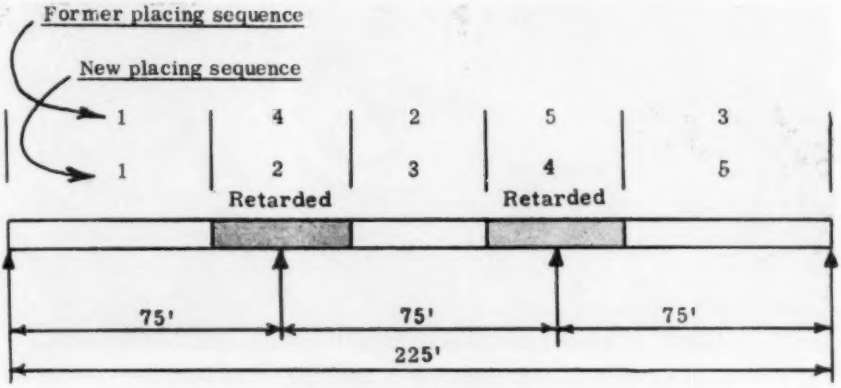


Fig. 9—Concreting sequence for continuous bridge deck

procedure for concreting the deck on such a bridge is to place the concrete in alternate panels, placing the concrete first in the positive moment areas (Fig. 9). Such a method is expensive, since the contractor has to wait until the concrete gains strength and then bring back his screeds and finishing equipment to place the alternate panels. Also, in such instances, the joint spacing is rather close and causes a maintenance problem. This method, at its best, is not perfect. If the concrete were retarded sufficiently, deflection would take place when the concrete is still plastic and could deform. No cracking would occur, joint spacing would be normal, and cost would be reduced. This method has been used successfully on a number of bridges, including the Newark Bay Bridge.

**Composite bridges**

Bridges utilizing composite construction are designed so that tensile stresses are carried by a girder and compressive stresses by a concrete slab (Alpha system). Before the deck is concreted, the bridge has no slab to take up compressive stresses. As concrete is placed, the girders will deflect. If the concrete loses slump or sets before the entire span has been concreted, deflection of the girders due to the weight of freshly placed concrete will cause the stiffened or set concrete to crack.

If the setting time of the concrete is sufficiently retarded the entire span can be placed before any setting or stiffening occurs. Deflection will take place when the concrete is plastic and cracking will be eliminated. This method has been used on bridges on the Garden State Parkway and the Maine and Connecticut Turnpikes.

Where cast in place, post-tensioned composite bridges are constructed, the setting time of the concrete used in the girders can be retarded sufficiently so that the concrete in the girders remains plastic until the slab is cast. In this manner, the entire bridge, girders and slabs, can be cast as a monolithic

Fig. 10—Wolcott Avenue Bridge. Deck slab is cast after beams have been revibrated. Then vibrators are inserted through fresh deck slab and 12 in. into beams, thus unifying lifts into one monolithic structure

unit, and revibrated together (Fig. 10). Casting in this manner eliminates the necessity for shear connectors and eliminates the cold joint normally found between girders and slabs. The Gorgas Lane Bridge in Philadelphia, and the Wolcott Avenue-Windsor Bridge in New Haven, Conn.,[6] were constructed in this manner.

### Slip form work

Slip form work requires careful control. Forms are supported by steel rods embedded in hardened but freshly placed concrete and are moved up at short time intervals. If the concrete sets too slowly, cave-ins will result, since the plastic concrete has not gained sufficient strength to prevent buckling of the steel rods which support the formwork and working platforms. If the concrete sets too fast, forms might bind and cold joints might occur with resultant leakage and maintenance expense during the life of the building. Slip form operations must proceed on a 24-hr basis. Since the temperature drops at night, the concrete will set slower at night and faster during the day. By keeping the set perfectly uniform, retarding the concrete more during the day, less at night, formwork can rise at a uniform rate. A suitable set retarder will cause concrete to gain strength faster after setting has taken place.

### Central mixed concrete

The use of central mixed concrete for highway or other construction is often limited to relatively short hauls. Some specifications permit only 20 min between mixing and placing, others up to 1½ hr. Concrete must be placed before it has stiffened so that water will not have to be added for proper workability. Stiffening can be delayed, time intervals lengthened, and use of ready-mixed concrete expanded with resultant economies by the judicious use of a set retarding agent.

### Winter concreting

Where there is danger of freezing of unhardened concrete, accelerators may be desirable; by speeding set, heat development, and early strength the concrete is out of the danger zone sooner.[4]

## CONCLUSION

There are many factors, physical as well as chemical, which influence the setting time of concrete. Setting time and hardening rate should be adjusted to secure optimum quality and economy of concrete construction under varying job conditions. This can be done by the use of varying proportions of single purpose admixtures that either accelerate or retard the setting time of concrete.

## ACKNOWLEDGMENT

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**Discussion of this paper should reach ACI headquarters in triplicate by Apr. 1, 1959, for publication in the Part 2, September 1959 JOURNAL.**

**Title No. 55-51** presents an investigation of flexural bond in beams pretensioned with seven-wire strand of ¼, ⅜, and ½ in. diameter. Primary object was to clarify the factors affecting bond action and strength, and to study the influence of certain variables on strand slip. The principal factor investigated was variation of strand embedment length.

The experimental results are discussed in detail, and criteria are presented for adequate design with respect to general bond slip of pretensioned strand. A hypothetical shape for the flexural bond stress wave immediately before bond slip is derived from the experimental results reported.

# FLEXURAL BOND TESTS

## of pretensioned prestressed beams

NORMAN W. HANSON and PAUL H. KAAR

IN AMERICAN PRESTRESSING PRACTICE, pretensioning has developed more rapidly than post-tensioning and a dominating trend has developed in recent years toward use of the seven-wire prestressing strand. Associated with the use of strand for pretensioning, there has been an economy-dictated trend toward the use of increased diameter of strand. The ¼ in. diameter strand was in common use in 1950. In 1956, extensive use was made of the ⅜ in. diameter strand, and some projects have recently utilized the ½-in. strand.

At this time there is no generally accepted method of predicting the bond performance of pretensioned strand; the effect on bond of these strand size increases is not fully understood. Bond studies of pretensioned concrete beams have, with few exceptions, been confined to prestress transfer bond, rather than flexural bond. The exceptions were studies by Janney[1] on flexural bond of single wire, and by Thorsen[2] and Norby and Venuti[3] on the relationship of flexural to transfer bond of pretensioned tendons.

The work reported in this investigation was done in the Research and Development Laboratories of the Portland Cement Association in 1955-57. The study was undertaken to assist in answering the often repeated query, "What is the maximum diameter of pretensioned seven-wire strand which one can safely use in a particular beam?" This question cannot be answered adequately without knowledge of the conditions under which bond failure may take place. Thus, the objectives of this investigation were primarily to

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develop a theory of bond action predicting ultimate strength in bond, and secondarily to study the influence of various factors on the bond performance of prestressing strand.

### Scope of tests

The embedment length and diameter of strand were the principal variables investigated with respect to their influence on the bond performance of pretensioned prestressed beams. The influence of reinforcement percentage and reduction in concrete strength were investigated in a more limited manner. In addition, pilot tests checked on the influence of strand surface condition and the effect of embedded end anchorages on pretensioned strand.

### Notation

The notation proposed by the Joint ACI-ASCE Committee 323, Prestressed Reinforced Concrete is used where applicable:

$A_s$ = area of tension reinforcement
$a$ = one-half the distance between layers of strand
$b$ = width of rectangular section
$c$ = depth of compression zone at ultimate moment
$d$ = effective depth
$\Sigma o$ = total circumferential area per in. length of beam
$\epsilon_{cu}$ = tensile strain in concrete at level of reinforcement at ultimate moment
$\epsilon_{se}$ = tensile strain in reinforcement due to effective prestress
$\epsilon_{su}$ = tensile strain in reinforcement at ultimate moment
$\epsilon_u$ = ultimate compressive strain in concrete
$F = \dfrac{\epsilon_{su} - \epsilon_{se}}{\epsilon_{cu}}$ = ratio of strain increase in the steel relative to strain increase in the concrete at the steel level
$f_c'$ = compressive strength of 6 × 12-in. cylinders
$f_{sb}$ = stress in reinforcement at general bond slip
$f_{se}$ = effective prestress in reinforcement
$f_{su}$ = stress in reinforcement at ultimate moment
$f_{vp}$ = yield strength of strand at 1 percent offset
$k_1k_3$ = coefficient determining average concrete stress in the compression zone at ultimate moment
$k_2$ = coefficient determining position of internal compressive force $C$
$l$ = length of shear span from concentrated load to nearest support reaction
$l_t$ = length of transfer zone
$l_u$ = embedment length of strand = distance from section of maximum steel stress to free strand end
$M_c$ = cracking moment

$M_{flex}$ = calculated ultimate flexural moment

$M_{test}$ = measured moment at ultimate strength

$M_{bond}$ = measured moment at general bond slip

$p = A_s/bd$ = steel ratio

$q_p$ = tension reinforcement index $= pf_{sp}/f_c'$

$u_a$ = average bond stress over entire embedment length as defined by Eq. (10)

$u_a'$ = average bond stress between end of transfer zone and a section of maximum strand stress as defined by Eq. (A1)

## ANALYTICAL CONCEPTS OF BOND ACTION

In this discussion of bond in pretensioned beams, much use is made of Janney's paper[1] on the same subject.

### Nature of bond

Bond in pretensioned concrete beams is of two types—transfer bond and flexural bond. Transfer bond utilizes a part of the available tensile strength of the steel to establish compression in the concrete. Flexural bond results from the action of external loads on beams. After cracking, the increase in steel stress above effective prestress develops flexural bond stress between the steel and concrete.

*Transfer bond*—Prestress transfer bond exists near beam ends after the load in the tensioned strand has been transferred to the concrete member. The length over which this transfer is made is termed the prestress transfer length, and depends mainly on the amount of prestress, surface condition of the strand, the strength of the concrete, and the method of steel stress release, which in these tests was gradual. Three factors which contribute to bond performance are adhesion between concrete and steel, friction between concrete and steel, and mechanical resistance between concrete and steel. In the transfer zone, reduction in the tensile strain of the steel does not equal the compressive strain in the concrete at the same point. There is relative movement of steel and concrete, and accordingly adhesion cannot contribute to prestress transfer. Friction is considered to be the principal agent causing stress transfer from pretensioning steel to concrete. As the tension in the strand is released, the strand diameter tends to increase, thus producing high radial pressure against the concrete, which in turn produces high frictional resistance in the transfer zone. Mechanical resistance probably contributes little to prestress transfer in the case of individual smooth wires, but it may be a factor of some significance in the case of strand.

*Flexural bond*—Flexural bond of significant magnitude exists only after the concrete beam has been loaded to cracking. When the concrete cracks, the bond stress in the immediate vicinity of the cracks rises to some limiting stress, slip occurs over a small portion of the strand length adjacent to the cracks, and the bond stress near the cracks is then reduced to a low value. With continued increase in load, the high bond stress progresses as a wave from the original cracks toward the beam ends. The bond stress remaining

behind the wave is always lower than the maximum value at the peak of the bond stress wave.

If the peak of the high bond stress wave reaches the prestress transfer zone, the increase in steel stress resulting from the bond slip decreases the strand diameter, reduces the frictional bond resistance, and precipitates general bond slip. Following loss of frictional resistance, mechanical resistance is the only factor which can contribute to bond between concrete and steel. If the beam is prestressed with clean smooth wire this resistance will be slight, and the beam will quickly collapse after general slippage has occurred. If the wire is rusted, the resistance to slippage will be greater. If the beam is prestressed with strand, the helical shape of the individual wires will provide sufficient mechanical resistance that the beam can support additional load even after slip of the strand at the beam ends. This mechanical resistance of strand to continued slip will be discussed later.

### Significance of the *F*-value

The ratio of steel flexure strain to apparent concrete flexure strain at the level of the steel has been designated $F$[4]: in an uncracked pretensioned prestressed beam the flexural strains in the steel and in the concrete at the level of the steel will be the same and $F$ will equal 1.0. When a pretensioned beam cracks, a certain amount of local bond slip must occur on each side of each crack. From cracking load onward, the value of $F$ will depend on the amount of local bond slip adjacent to the cracks. Where considerable local bond slip occurs, $F$ can have values considerably below 1.0.

This concept of the relationship between steel strains and concrete strains is a useful one in interpreting beam behavior, but the variable nature of $F$ for strand-reinforced beams seems almost to preclude its determination.

The ultimate flexural strength calculations for the test beams are based on an $F$ of 1.0. Analysis of data[5] has confirmed this as a reasonable value for good bond conditions with ¼ to ½-in. tendons.

## COMPUTATION OF ULTIMATE FLEXURAL STRENGTH

In analyzing the test data it was desirable to use the more precise methods of computing ultimate strength rather than simplified design equations, so that the relationships between test values and theoretical values might be determined as accurately as possible.

All ultimate moments given as $M_{flex}$ in this paper are calculated using the ultimate strength factors, $k_1$, $k_2$, $k_3$, and $\epsilon_u$ presented by Hognestad, Hanson, and McHenry[6] in a paper concerning the stress distribution in the compression zone of structural concrete flexural members. The ultimate strength factors were expressed as functions of concrete strength as follows:

$$k_1 k_3 = \frac{3900 + 0.35 f_c'}{3200 + f_c'} \quad \ldots\ldots (1)$$

$$k_3 = 0.50 - \frac{f_c'}{80{,}000} \quad \ldots\ldots (2)$$

$$\epsilon_u = 0.004 - \frac{f_c'}{6.5 \times 10^6} \quad \ldots\ldots(3)$$

Using these factors and a value of $F = 1.0$, the steel stress at ultimate flexural strength $f_{su}$ was calculated by successive approximations as described by Janney, Hognestad, and McHenry[5]. Here, for beams containing steel at one level, we have from requirements of compatibility of strains:

$$\frac{c}{d} = \frac{F\epsilon_u}{\epsilon_{su} - \epsilon_{se} + F\epsilon_u} \quad \ldots\ldots(4)$$

From condition of equilibrium of concrete and steel forces:

$$c = \frac{f_{su} A_s}{k_1 k_3 f_c' b} \quad \ldots\ldots(5)$$

The flexural ultimate moment is given by:

$$M_{flex} = f_{su} A_s (d - k_2 c) \quad \ldots\ldots(6)$$

In this calculation, $\epsilon_{su}$ is first assumed, and $f_{su}$ is then computed using Eq. (4) and (5). The values of $\epsilon_{su}$ and $f_{su}$ so obtained are checked for compatibility with the stress-strain curve for the strand involved. Several trials may be necessary to establish this compatibility. The final value of $f_{su}$ so obtained is substituted in Eq. (5) to give the neutral axis depth $c$, which in turn is substituted in Eq. (6) to obtain the ultimate moment of resistance of the section.

For beams containing steel placed in two layers, subscripts $t$ and $b$ are added to the steel notation to denote top and bottom layer. The strain compatibility equations are then:

$$\frac{c}{d} = \frac{F\epsilon_u \left(1 - \frac{a}{d}\right)}{\epsilon_{sut} - \epsilon_{se} + F\epsilon_u} = \frac{F\epsilon_u \left(1 + \frac{a}{d}\right)}{\epsilon_{sub} - \epsilon_{se} + F\epsilon_u} \quad \ldots\ldots(7)$$

The condition of equilibrium of forces leads to:

$$c = \frac{f_{sub} A_{sb} + f_{sut} A_{st}}{k_1 k_3 f_c' b} \quad \ldots\ldots(8)$$

The flexural moment is then given by the equation:

$$M_{flex} = f_{sub} A_{sb} (d_b - k_2 c) + f_{sut} A_{st} (d_t - k_2 c) \quad \ldots\ldots(9)$$

The values of $f_{su}$ and $\epsilon_{su}$ for the two layers of steel are obtained by the method of successive approximations as before. Eq. (8) and (9) are then used to calculate the depth of the neutral axis and the ultimate moment of resistance.

## TEST SPECIMENS

### Outline of tests

The test program involved 47 beam tests divided into four series as shown in Table 1:

*Series 1*—Eighteen prestressed beams containing clean, smooth pretensioned strands were tested over various shear-span lengths covering a wide range. These beams contained, separately, either ¼, ⅜, or ½ in. diameter strand. These tests were conducted to evaluate the effect of strand diameter and embedment length on bond performance.

**TABLE 1—BEAM PROPERTIES**

| Beam series and No. | Strand No. and size, in. | Beam width, $b$, in. | Beam effective depth, $d$, in. | Shear span, $l$, in. | Embedment length, $l_e$, in. | Concrete strength, $f'_c$, psi | Effective prestress, $f_{se}$, ksi | Percentage steel, $p$ |
|---|---|---|---|---|---|---|---|---|
| 1–1 | 4 ¼ | 6.0 | 8.61 | 21 | 27 | 6040 | 141.0 | 0.279 |
| 1–2 | 4 ¼ | 6.0 | 8.54 | 31 | 37 | 6620 | 141.0 | 0.281 |
| 1–3 | 4 ¼ | 6.0 | 8.56 | 36 | 42 | 7800 | 141.0 | 0.280 |
| 1–4 | 4 ¼ | 6.0 | 8.76 | 42 | 48 | 6040 | 141.0 | 0.274 |
| 1–5 | 4 ¼ | 6.0 | 8.67 | 84 | 90 | 6040 | 141.0 | 0.277 |
| 1–6 | 4 ¼ | 6.0 | 8.68 | 168 | 174 | 5980 | 141.0 | 0.276 |
| 1–7 | 3 ⅜ | 6.0 | 8.56 | 21 | 27 | 5730 | 129.7 | 0.467 |
| 1–8 | 3 ⅜ | 6.0 | 8.63 | 42 | 48 | 5730 | 129.7 | 0.463 |
| 1–9 | 3 ⅜ | 6.0 | 8.65 | 84 | 90 | 5730 | 129.7 | 0.462 |
| 1–10 | 3 ⅜ | 6.0 | 8.60 | 168 | 174 | 5130 | 144.6 | 0.465 |
| 1–11 | 2 ½ | 6.0 | 8.40 | 21 | 33 | 5540 | 148.0 | 0.570 |
| 1–12 | 2 ½ | 6.1 | 8.50 | 30 | 36 | 5600 | 132.0 | 0.555 |
| 1–13 | 2 ½ | 6.1 | 8.50 | 34 | 40 | 6300 | 132.0 | 0.555 |
| 1–14 | 2 ½ | 6.0 | 8.41 | 42 | 54 | 5540 | 139.0 | 0.569 |
| 1–15 | 2 ½ | 6.1 | 8.44 | 60 | 66 | 5600 | 132.0 | 0.558 |
| 1–16 | 2 ½ | 6.1 | 8.31 | 71 | 77 | 5600 | 132.0 | 0.566 |
| 1–17 | 2 ½ | 6.1 | 8.69 | 84 | 90 | 5090 | 132.0 | 0.543 |
| 1–18 | 2 ½ | 6.1 | 8.40 | 168 | 174 | 5090 | 132.0 | 0.561 |
| 2–1 | 4 ⅜ | 6.0 | 8.62 | 54 | 60 | 3700 | 124.9 | 0.618 |
| 2–1R* | 4 ⅜ | 6.0 | 8.47 | 54 | 60 | 3700 | 124.9 | 0.630 |
| 2-1A† | 4 ⅜ | 6.0 | 8.41 | 54 | 60 | 3700 | 124.9 | 0.634 |
| 2–2 | 4 ⅜ | 6.0 | 8.62 | 54 | 60 | 5420 | 119.4 | 0.618 |
| 2–2R | 4 ⅜ | 6.0 | 8.44 | 54 | 60 | 5420 | 119.4 | 0.632 |
| 2-2A | 4 ⅜ | 6.0 | 8.44 | 54 | 60 | 5420 | 119.4 | 0.632 |
| 2–3 | 4 ⅜ | 6.0 | 8.69 | 54 | 60 | 7230 | 121.2 | 0.614 |
| 2–3R | 4 ⅜ | 6.0 | 8.56 | 54 | 60 | 7230 | 121.2 | 0.623 |
| 2–3A | 4 ⅜ | 6.0 | 8.45 | 54 | 60 | 7230 | 121.2 | 0.631 |
| 3–1 | 2 ¼ | 4.0 | 5.69 | 36 | 40 | 5020 | 146.0 | 0.316 |
| 3–2 | 4 ¼ | 4.0 | 5.66 | 36 | 40 | 5720 | 142.0 | 0.637 |
| 3–3 | 6 ¼ | 4.0 | 5.86 | 36 | 40 | 5900 | 113.0 | 0.921 |
| 3–4 | 2 ⅜ | 6.0 | 8.44 | 54 | 60 | 5300 | 134.0 | 0.316 |
| 3–5 | 2 ⅜ | 6.0 | 8.37 | 54 | 60 | 5400 | 131.0 | 0.318 |
| 3–6 | 4 ⅜ | 6.0 | 8.53 | 54 | 60 | 5300 | 136.0 | 0.625 |
| 3–7 | 4 ⅜ | 6.0 | 8.54 | 54 | 60 | 5900 | 119.0 | 0.624 |
| 3–8 | 6 ⅜ | 6.0 | 8.53 | 54 | 60 | 5450 | 132.5 | 0.938 |
| 3–9 | 6 ⅜ | 6.0 | 8.07 | 54 | 60 | 5700 | 127.0 | 0.991 |
| 3–10 | 2 ½ | 8.0 | 11.47 | 72 | 80 | 6450 | 120.0 | 0.313 |
| 3–11 | 4 ½ | 8.0 | 11.40 | 72 | 80 | 6050 | 135.0 | 0.631 |
| 3–12 | 6 ½ | 8.0 | 11.33 | 72 | 80 | 5420 | 132.0 | 0.963 |
| 4–1 | 1 ⅜ | 4.0 | 5.56 | 28 | 34 | 5750 | 128.0 | 0.359 |
| 4–1R | 1 ⅜ | 4.0 | 5.56 | 28 | 34 | 5750 | 128.0 | 0.359 |
| 4–2 | 1 ½ | 4.0 | 5.59 | 28 | 32 | 5720 | 141.0 | 0.643 |
| 4–2R | 1 ½ | 4.0 | 5.50 | 28 | 32 | 5720 | 139.0 | 0.654 |
| 4–3 | 1 ½ | 6.0 | 8.29 | 42 | 48 | 5430 | 147.0 | 0.289 |
| 4–3R | 1 ½ | 6.0 | 8.40 | 42 | 48 | 5500 | 141.0 | 0.285 |
| 4–4 | 2 ½ | 6.0 | 8.38 | 42 | 48 | 5350 | 142.0 | 0.572 |
| 4–4R | 2 ½ | 6.0 | 8.45 | 42 | 48 | 5350 | 147.0 | 0.567 |

*R indicates use of rusted strand.
†A indicates use of strand anchor embedded in concrete at beam ends.

*Series 2*—In nine beams with ⅜-in. strand, tested in three groups, the concrete strengths of the three groups were 3700, 5420, and 7230 psi. Within each group, one beam contained a rusted strand, one a clean, smooth strand, and one a clean, smooth strand with a "Strand-vise" anchor within the concrete at each end. These tests were designed to evaluate primarily effects of concrete strength on bond performance.

*Series 3*—Twelve test specimens comprising three sets of geometrically similar beams ($b$ and $d$ variable, but $b/d$ constant) were prestressed with clean, smooth strands. The sets were reinforced with ¼, ⅜, and ½-in. strands, one strand size in each set, and the number of strands per beam was two, four, and six. The tests in this series were intended to demonstrate primarily the influence on bond performance of variation of reinforcement percentage.

*Series 4*—Eight beams were prestressed with ⅜ in. or ½ in. diameter strands. The beams were made in pairs, one with clean, smooth strand and one with rusted strand. The tests of this group together with Series 2, were expected to indicate possible effects on beam performance of strand surface condition.

## AND TEST RESULTS

| $q_p = \frac{p f_{pu}}{f_c'}$ | Cracking moment, $M_c$, in.-kips | $M_{bond}$ at bond slip, in.-kips | $M_{test}$ at ultimate strength, in.-kips | Calculated flexural ultimate, $M_{flex}$, in.-kips | $\frac{M_{bond}}{M_{flex}}$ | $\frac{M_{test}}{M_{flex}}$ | Mode of failure‡ |
|---|---|---|---|---|---|---|---|
| 0.121 | 207.2 | 287.0 | 337.4 | 296.9 | 0.97 | 1.14 | B |
| 0.111 | 151.0 | 278.1 | 313.2 | 295.2 | 0.94 | 1.06 | B |
| 0.094 | 146.2 | 299.2 | 309.1 | 299.2 | 0.99 | 1.03 | B |
| 0.118 | 206.1 | — | 322.6 | 302.4 | — | 1.07 | F |
| 0.120 | 202.6 | — | 322.3 | 298.1 | — | 1.08 | F |
| 0.121 | 190.0 | — | 302.4 | 299.4 | — | 1.01 | F |
| 0.196 | 241.9 | 380.4 | 423.0 | 411.4 | 0.93 | 1.03 | B |
| 0.195 | 239.7 | 428.6 | 428.6 | 417.1 | 1.03 | 1.03 | B |
| 0.194 | 238.3 | — | 464.1 | 464.1 | — | 1.11 | F |
| 0.195 | 227.6 | — | 439.3 | 408.6 | — | 1.07 | F |
| 0.261 | 292.0 | 399.7 | 418.0 | 488.0 | 0.82 | 0.86 | B |
| 0.251 | 200.5 | 295.0 | 429.5 | 493.2 | 0.60 | 0.87 | B |
| 0.223 | 269.8 | 380.3 | 450.3 | 505.9 | 0.75 | 0.89 | B |
| 0.260 | 294.4 | 426.7 | 481.3 | 486.0 | 0.88 | 0.99 | B |
| 0.252 | 248.5 | 439.0 | 457.0 | 488.4 | 0.90 | 0.94 | B |
| 0.256 | 216.4 | 440.3 | 455.4 | 473.7 | 0.93 | 0.98 | B |
| 0.270 | 243.1 | — | 503.4 | 495.0 | — | 1.02 | F |
| 0.279 | 256.8 | — | 474.9 | 470.0 | — | 1.01 | F |
| 0.402 | 289.0 | — | 504.6 | 461.6 | — | 1.09 | F |
| 0.410 | 289.9 | — | 493.8 | 453.7 | — | 1.09 | F |
| 0.413 | 289.9 | — | 496.5 | 451.4 | — | 1.10 | F |
| 0.298 | 314.2 | 451.9 | 492.4 | 535.0 | 0.84 | 0.92 | B |
| 0.298 | 295.3 | — | 549.1 | 526.2 | — | 1.04 | F |
| 0.298 | 300.7 | 503.2 | 507.3 | 517.6 | 0.97 | 0.98 | B |
| 0.204 | 346.6 | 603.1 | 609.9 | 538.7 | 1.12 | 1.13 | B |
| 0.207 | 326.3 | 603.1 | 607.2 | 532.3 | 1.13 | 1.14 | B |
| 0.210 | 327.7 | 603.1 | 608.5 | 523.8 | 1.15 | 1.16 | B |
| 0.160 | 59.3 | 88.1 | 94.4 | 92.5 | 0.95 | 1.02 | B |
| 0.283 | 95.3 | 172.7 | 185.3 | 163.1 | 1.06 | 1.13 | B |
| 0.397 | 115.1 | 210.6 | 210.6 | 204.0 | 1.04 | 1.04 | B |
| 0.155 | 207.2 | 308.5 | 312.5 | 314.0 | 0.98 | 0.99 | B |
| 0.138 | 151.9 | 259.9 | 259.9 | 285.0 | 0.91 | 0.91 | B |
| 0.307 | 326.0 | 529.9 | 543.4 | 546.0 | 0.97 | 1.00 | B |
| 0.248 | 211.3 | 454.3 | 465.1 | 463.0 | 0.98 | 1.00 | B |
| 0.448 | 417.8 | — | 695.9 | 666.0 | — | 1.05 | F |
| 0.409 | 332.8 | 643.3 | 643.3 | 561.0 | 1.15 | 1.15 | B |
| 0.123 | 466.9 | 589.3 | 681.1 | 766.0 | 0.77 | 0.89 | B |
| 0.264 | 801.7 | — | 1330.9 | 1296.5 | — | 1.03 | F |
| 0.450 | 1035.7 | 1521.7 | 1602.7 | 1505.0 | 1.01 | 1.07 | B |
| 0.147 | 64.8 | 79.8 | 79.8 | 92.0 | 0.89 | 0.89 | B |
| 0.147 | 62.7 | 78.7 | 104.9 | 92.0 | 0.86 | 1.16 | B |
| 0.285 | 96.9 | 99.7 | 117.9 | 155.5 | 0.64 | 0.76 | B |
| 0.290 | 92.7 | 148.7 | 148.7 | 151.0 | 0.98 | 0.98 | B |
| 0.135 | 176.8 | 218.8 | 294.4 | 277.0 | 0.79 | 0.79 | B |
| 0.131 | 189.3 | 302.8 | 302.8 | 280.5 | 1.08 | 1.08 | B |
| 0.271 | 281.8 | 363.7 | 425.8 | 477.0 | 0.76 | 0.89 | B |
| 0.269 | 296.8 | 551.6 | 551.6 | 486.0 | 1.13 | 1.13 | B |

‡B designates bond slip to end of strand either before or at ultimate moment.
F designates flexural failure without bond slip to end of strand.

### Materials

All beams were cast by internal vibration of a 2-in. slump concrete containing 1½-in. maximum size aggregate. The cement was a blend of four brands of Type I portland cement. Moist curing under burlap for the first 3 days was followed by storage at approximately 70 F and 50 percent relative humidity. Concrete cylinder strengths for each beam specimen are given in Table 1, based on tests of at least three 6 × 12-in. cylinders cast and cured with each beam specimen, or each group of beams cast simultaneously in the pretensioning bed.

The prestressing steel was seven-wire strand of ¼, ⅜, and ½ in. nominal diameter. Nominal cross-sectional areas and circumferences are given in Table 2.

The stress-strain curves to 1 percent offset are shown in Fig. 1. The stress is tension force divided by nominal strand cross section, and the strain was measured by three SR-4 gages mounted on separate single wires of the strand. For purposes of beam test analyses, strand strain measured in beams by SR-4 gages was converted into stress by the applicable curve of Fig. 1. Note that there are two different steels used in the beams with ¼-in. strand and three different steels used in the beams with ⅜-in. strand.

**TABLE 2—STRAND PROPERTIES**

| Strand size, in. | Nominal circumference, 4/3 $\pi d$, in. | Nominal cross-sectional area, sq. in. |
|---|---|---|
| ¼ | 1.05 | 0.036 |
| ⅜ | 1.58 | 0.080 |
| ½ | 2.10 | 0.144 |

All strand was cleaned of surface oil by washing in carbon tetrachloride. The rusted strands used in special beam tests were obtained by exposing the strand to a moist environment for approximately 1 week. The rust coating so obtained was uniform without pitting.

## Instrumentation

As bond failure is progressive rather than instantaneous, the test beams were instrumented to record the continuous development of flexural strains and the subsequent evidence of bond slip. Bonded SR-4 type A-12-2 strain gages were cemented along the helical individual wires of the twisted strand. The beams in Series 3 were instrumented with gages at 12, 15, and 20-in. intervals for ¼, ⅜, and ½-in. strand size, respectively. To minimize reduction of surface area available for bond of any one strand, the gages were placed alternately on adjacent strands. Various gage spacings were used in beams of other series.

In addition to their use during the testing of the specimens, the strain gages were used to gage the proper strand tension before casting of the beams. Strain gage readings were taken immediately before and after tensioning, and also before and after stress release to determine immediate losses. These losses at the beam center ranged from 1.7 to 7.3 percent of the initial wire strain.

Dial indicators reading to 0.001 in. were used to measure beam deflections. Dial gages were also mounted at both ends of each beam to detect end slip of the projecting strand ends relative to the concrete.

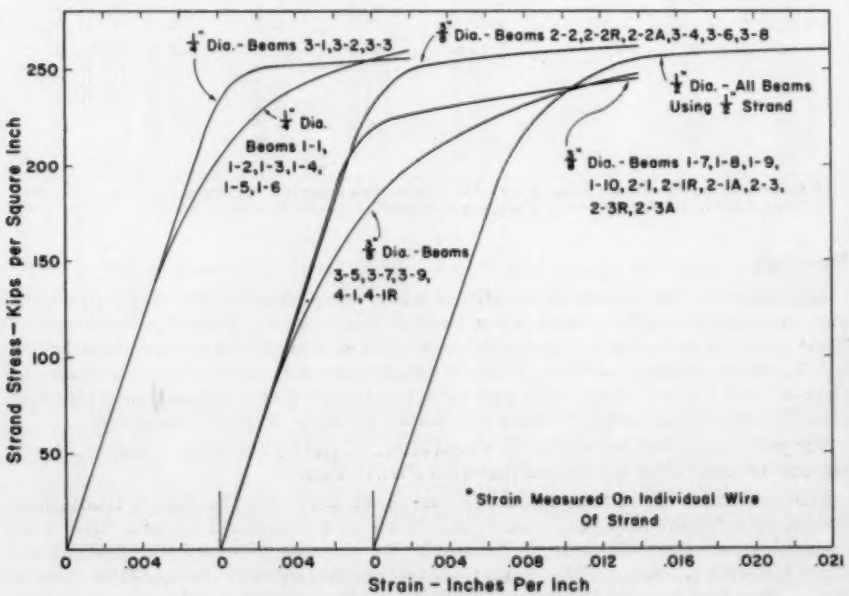


Fig. 1—Stress-strain curves of strand

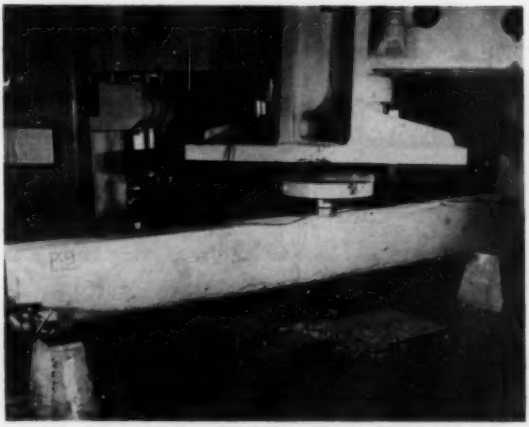

Fig. 2—Typical test arrangement

### Test procedure

All beams were tested to failure in a hydraulic testing machine as shown in Fig. 2. Beams of Series 2 and 3 were loaded at midspan through either a ball or roller arrangement. All other beams were loaded at two points through rollers spaced twice the beam width on either side of midspan. The beams were supported on concrete piers with a free roller at one end and a pivot at the other end.

The strand tension was released at a concrete strength of 4500 psi except for the lower strength specimens of Series 2, which were released at a strength of 3500 psi. The beams were tested at the strengths shown in Table 1.

The individual beam tests were generally conducted in about 30 min, with a continuously increasing load. The steel strains as indicated by the SR-4 gages were recorded continuously, and dial gage readings of deflection and strand slip were made without interrupting the loading.

## TEST RESULTS

### Typical beam behavior

As an introduction to the discussion of test results, the test of a typical specimen of this project will be described in detail—Beam 3-10 of Series 3 shown in Fig. 3. Electrical resistance gages were placed 40 in. apart on the two ½ in. diameter strands, staggered so that there were three 20-in. intervals on either side of midspan. Since the 160 in. long beam was tested on a 12-ft span, the gages were spaced to within 20 in. of the beam end. Strains were continuously recorded during the entire test from zero load to ultimate strength. A load was applied at midspan of the beam through a spherical loading block bearing on a 4 in. wide plate.

Test results are plotted in Fig. 3. Strains have been converted to stresses, using the applicable stress-strain curve of Fig. 1. Strains before cracking are relatively small; they correspond to a stress increase above the prestress of less than 5000 psi, and flexural bond stresses are only 5 to 6 psi. These stresses are typical, however, of service conditions since service loads in pretensioned beams or slabs are ordinarily less than cracking loads.

In this test, with the beam center loaded, the initial crack formed at Gage 4, the center gage. At the cracking load of 12,450 lb, the steel strain at this crack location increased sharply and continued to increase until the end of the test. As the loading increased, the wave of high strains, caused by the progressing bond wave, enveloped Gages 5 and 3 at either side of the center. The bond wave intersected Gage 5 at 13,100 lb and Gage 3 at 15,400 lb load. The strand stress increases caused by the bond wave did not originate only with the midspan crack. Other cracks formed at either side of midspan during the test, and strain increases were caused by waves originating with the midspan crack and nearby cracks.

At a load of 15,500 lb, stress in Gage 5 began to decrease even though the beam load was increasing. This indicated bond slip; the initial slippage of the strand relative to the concrete caused some stress relaxation in the strand. This stress relaxation at slip reduced the strand stress only slightly, for fric-

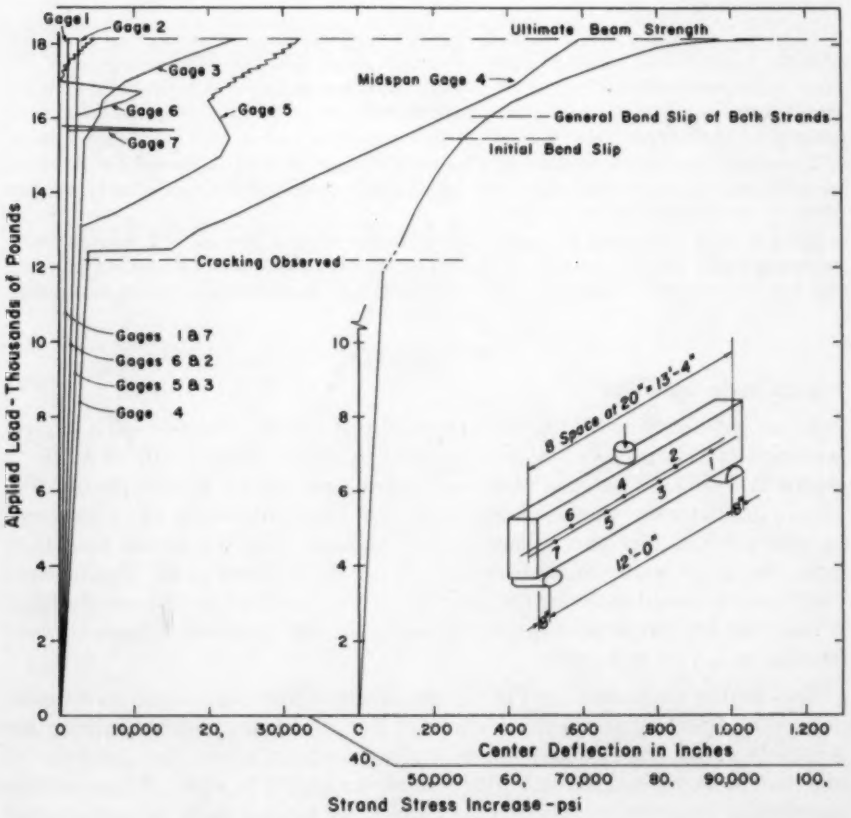


Fig. 3—Load versus strand stress increase and deflection, Beam 3-10

tional bond and mechanical resistance of the twisted strand to further movement in the beam replaced the now-destroyed initial adhesive bond. From this load to complete failure the strain trace of Gage 5, as recorded, moved in small jerks, showing the sawtooth trace indicative of the development of a mechanical interlock resisting slip.

Shortly after the bond slip indication at Gage 5, Gage 7, 40 in. away, showed a large strain increase followed by a reduction to a strain below the prestress level. The bond wave at this time had traveled to within 20 in. of the beam end. Since this location was within the prestress transfer length, resistance to flexural bond stress was only slight.

The same bond phenomena can be traced on the other of the two strands. At a load of 16,100 lb, slip occurred at Gage 6. This gage was near the prestress transfer zone, and bond slip here was not as sudden as at Gage 7 well within the transfer zone.

At the other end of the beam, it is apparent from the record of Gages 1 and 2 that the strands did not fail in bond. The bond wave traveled from the center to beyond Gage 3, but did not reach Gages 1 and 2.

General bond slip of both strands was indicated at a load of 16,100 lb by slip measured by dial gages at the protruding strand ends. Thereafter, the beam was loaded to 18,150 lb before crushing of the concrete at the top of the beam took place at midspan. Midspan deflections, shown in Fig. 3, increased rapidly after bond failure.

This characteristic beam behavior shows that seven-wire strand, in contrast to smooth individual wires, can develop additional beam strength even after general bond slip to the beam end has occurred. This result of mechanical bond resistance is an extremely important characteristic of strand performance.

### Discussion of test results

*Interrelation of variables*—If a strand is to be stressed to fracture at ultimate load of a beam, then there is a critical embedment length for each size of strand, which must be provided if bond slip is to be avoided. However, if a beam contains a high percentage of steel, or if the concrete strength is low, then flexural failure may occur by crushing of the concrete while the steel is stressed below its ultimate strength. In this case bond slip will not necessarily occur even if the embedment length provided is less than the critical length needed for the size of strand used to develop the ultimate strand strength. Thus concrete strength, percentage of steel, and embedment length are interrelated in design.

*Mode of failure*—Thirteen of the beams failed in flexure without prior slippage of the strand along its entire embedment length. The remainder failed in flexure after a general bond slip of the strands. The moment sustained at general bond slip and the ultimate moment sustained were both of interest in this investigation. Comparison of moments sustained at general bond slip and at ultimate beam strength with the theoretical ultimate flexural strength is presented in Table 1.

*Strand embedment length*—The effect of variation of strand embedment length on the moment at general bond slip and on the ultimate moment of resistance of a given section, is illustrated in Fig. 4, in which $M_{test}$ is the measured moment at ultimate strength, $M_{bond}$ is the measured moment at general bond failure, and $M_{flex}$ is the calculated ultimate flexural moment.

Beams having an embedment length from load to beam end of 80 in. or more failed in flexure by crushing of the concrete after yielding of the steel, before general bond slip could occur. As the embedment length decreased, failure occurred at progressively lower moments due to slippage of the strands. Failure by slippage of the strands occurred in two stages, (a) initial general slip of the strand along its whole embedment length, and (b) destruction of the mechanical interlocking effect between the strand surface and the surrounding concrete. In the case of strands with short embedment lengths, a considerable increase of load on the beam was possible between these stages. It should be emphasized, however, that the loading was of the static type. If such beams were subject to dynamic loading after the first stage of general bond slip, it is possible that the increase in load to reach stage two would be significantly reduced. Until dynamic loading tests can be carried out to cover this point, it appears desirable for *design* purposes to rely only on attaining a moment of resistance corresponding to general bond slip.

One reason for the scatter of data in these tests is initial slip of the center wire of the strand, which remained undetected by the strain gages. In several

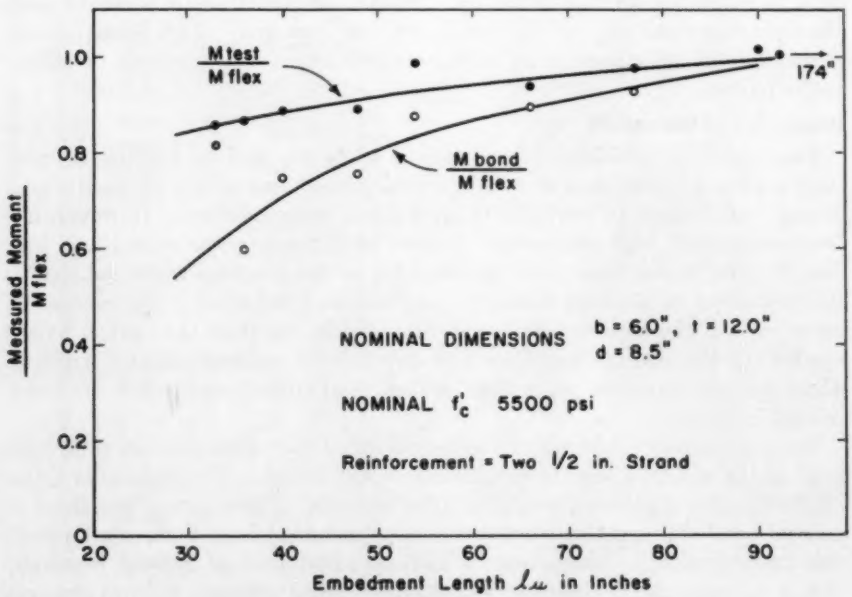


Fig. 4—Performance of beams with various embedment lengths

tests, the center wire had moved as much as 0.1 in. relative to the surrounding six wires at ultimate beam strength. In cases where the strand had been flame cut, thus fusing the strand end, the force exerted by the slipping center wire was in some cases enough to buckle the outside wires of the strand at the beam end.

TABLE 3—
PERFORMANCE OF SERIES 3 BEAMS

| Beams | Average steel percentage | Average $\frac{M_{bond}}{M_{flex}}$ | Average $\frac{M_{test}}{M_{flex}}$ |
|---|---|---|---|
| 3-1, 3-4, 3-5, 3-10 | 0.31 | 0.90 | 0.95 |
| 3-2, 3-6, 3-7, 3-11 | 0.62 | 1.01 | 1.04 |
| 3-3, 3-8, 3-9, 3-12 | 0.95 | 1.06 | 1.07 |

*Percentage of steel*—High steel percentages reduce the possibility of bond failure for a given embedment length because the steel stress at flexural failure is less in a beam with a high steel percentage than in a beam with a low $p$. It follows that, for a given embedment length, the average bond stress at flexural failure will be less in a beam with high steel percentage than in a beam with low steel percentage. The risk of exceeding the average bond stress at which general slip occurs for a particular embedment length is therefore greater with a beam having a low percentage of steel. The beams of Series 3 illustrate this point (see Table 3).

*Reduction of concrete strength*—The effect of reduction of concrete strength in a particular beam is to decrease the steel stress at flexural failure, with a correspondingly lower average bond stress over the embedment length. If the reduction in average bond stress due to drop in steel stress is greater than the reduction in the bond strength due to drop in concrete strength, then a failure due to general bond slip will be less likely in the beam with reduced concrete strength. Tests in Series 2 indicate that this is the case. These beams of identical cross section were tested with the same strand embedment length. The three beams of high strength concrete failed by general bond slip. Of the beams made of intermediate strength concrete, two failed by general bond slip and one by flexure. The low strength concrete beams all failed in flexure.

From these tests it appears that reduction of concrete strength has more influence on ultimate flexural strength than on ultimate bond strength.

*Strand surface condition*—The beams prestressed by rusted strand performed as well as, or better than, the beams prestressed by clean, smooth strand. Moment at general bond slip compared with beams having clean strand was improved as much as 53 percent in the case of Beam 4-2, although one beam (2-3) showed no significant improvement. The average improvement for all Series 4 beams with rusted strand was 30 percent. This confirms Janney's findings.[1] However, the danger of careless rusting allowing localized pitting should be remembered.

*Provision of end anchors*—A comparison of anchored and unanchored beams in Series 2 for three concrete strengths used shows little difference in performance. The end anchors did not become effective until slip had occurred along the entire embedment length, and they were therefore unable to delay

the onset of general bond slip. After the anchors became effective the beams acted as post-tensioned beams without bond, and their ultimate moments of resistance are no greater than those of the beams depending only on the mechanical interlocking effect between the strand and concrete after general bond slip.

**Analysis of test results**

*Strand size and embedment length*—These were the principal variables investigated with respect to their influence on the bond performance of pretensioned prestressed beams. In the case of these two variables a sufficient number of test results were obtained to warrant a detailed analysis. The influence of other less important variables has been discussed in general terms.

*Calculation of bond stresses*—At present it is not feasible to measure the magnitude and complex distribution of bond stresses along the entire length of a prestressing strand. However, to obtain suitable design criteria, we can calculate the average bond stress along the length of a strand from its free end to a section of maximum stress.

Consider the forces acting along the length of a prestressing strand between its free end and a section of maximum stress. The force due to tensile stress in the steel will then equal $f_s A_s$. The force due to bond stresses on the surface of the embedded strand will be given by $u_a\, l_u\, \Sigma o$. To satisfy the conditions of static equilibrium these forces must be equal: $u_a\, l_u\, \Sigma o = f_s A_s$. Therefore the average bond stress is given by:

$$u_a = \frac{f_s\, A_s}{l_u\, \Sigma o} \qquad (10)$$

Given the maximum steel stress at the time general bond slip occurs $f_{sb}$, and the embedment length $l_u$, we can calculate the average bond stress along the embedded length of strand:

$$u_a = \frac{f_{sb}\, A_s}{l_u\, \Sigma o} \qquad (11)$$

This average bond stress $u_a$ is a convenient measure of the bond performance of prestressing strand, and has the merit of having a real physical significance. The value of $u_a$ has been calculated for the various beams tested, and the results are plotted in Fig. 5.

For embedment lengths slightly greater than the transfer length, the combination of the peak zone of the flexural bond wave with the transfer bond stress region results in a high average bond stress just before general bond slip. For longer embedments, the flexural bond wave form includes a long "tail" stretching from a peak near the transfer zone to the section of maximum steel stress. The average bond stress just before general bond slip for long embedments is therefore less than the average bond stress at general bond slip for short embedments.

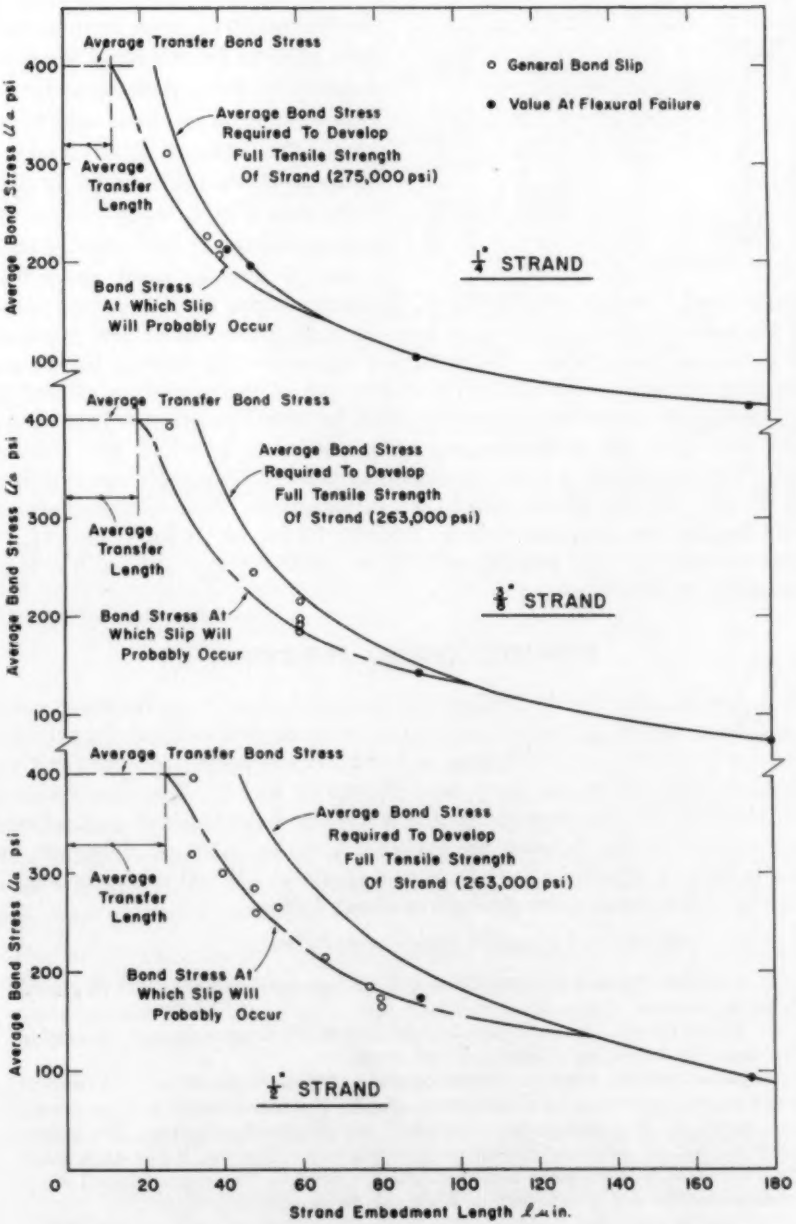
Average Transfer Bond Stress
Average Bond Stress Required To Develop Full Tensile Strength Of Strand (275,000 psi)
Average Transfer Length
Bond Stress At Which Slip Will Probably Occur
o General Bond Slip
• Value At Flexural Failure
¼" STRAND
Average Bond Stress Required To Develop Full Tensile Strength Of Strand (263,000 psi)
⅜" STRAND
½" STRAND
Average Bond Stress $u_a$ psi
Strand Embedment Length $\ell_a$ in.
0 20 40 60 80 100 120 140 160 180
100 200 300 400

Fig. 5—Relation of average bond stress $u_a$ to strand embedment length

**TABLE 4—STEEL STRESS AT GENERAL BOND SLIP, psi**

| Embedment length, in. | ¼-in. strand | ⅜-in. strand | ½-in. strand |
|---|---|---|---|
| 20 | 194,000 | 160,000 | — |
| 30 | 218,000 | 187,000 | 166,000 |
| 40 | 234,000 | 201,000 | 180,000 |
| 50 | 250,000 | 211,000 | 192,000 |
| 60 | 264,000 | 220,000 | 200,000 |
| 70 | — | 229,000 | 206,000 |
| 80 | — | 238,000 | 213,000 |
| 90 | — | 247,000 | 219,000 |
| 100 | — | 257,000 | 226,000 |
| 120 | — | — | 244,000 |
| 140 | — | — | 272,000 |

In the Appendix a possible shape for the flexural bond stress wave immediately prior to general bond slip is deduced from the experimental results. This wave form has been used to calculate the average bond stress at which general slip will probably occur in the case of each of the three strand sizes considered in this report. These values of average bond stress have been plotted as broken lines in Fig. 5. These theoretical lines are substantiated by the experimentally determined average bond stresses at general bond slip for individual beam tests. The solid line represents the average bond stress necessary to develop the full ultimate strength of the strand considered for any particular embedment length. The intersections of the broken and solid lines give the minimum required embedment length if the ultimate strength of the strand is to be developed by beam flexure before general bond slip occurs. For the strand used in this investigation, these minimum embedment lengths are approximately as follows: 70 in. for ¼-in. strand ($f_{vp}$ = 275,000); 106 in. for ⅜-in. strand ($f_{vp}$ = 263,000); and 134 in. for ½-in. strand ($f_{vp}$ = 263,000).

## SUGGESTED CRITERIA FOR DESIGN

It is usually desirable in prestressed concrete design that ultimate strength should be governed by flexure rather than by general bond slip. To this end, curves of steel stress at which general bond slip will probably occur for given strand embedment lengths have been plotted in Fig. 6. These curves have been obtained directly from the curves of average bond stress at general bond slip contained in Fig. 5, using the equation $f_s = (\Sigma o/A_s)\, u_a\, l_u$. They are applicable only to strand initially tensioned to about 150,000 psi, embedded in concrete with a compressive strength of about 5500 psi.

The proposed method of use in design is as follows:

1. Calculate the steel stress at ultimate flexural strength, assuming that no general bond slip occurs.
2. Check the embedment length of strand, that is, the distance from the free end of the strand to the section of maximum steel stress.
3. Read from Fig. 6 the maximum steel stress that can be developed in the embedment length provided for the chosen size of strand. If this stress exceeds by an acceptable margin the steel stress at flexural ultimate strength already calculated, then general bond slip will not occur and the section is satisfactory. However, if this stress is less than the calculated steel stress at flexural ultimate strength, then a general bond slip is probable and the section or the strand size should be modified to avoid this.

To supplement Fig. 6, Table 4 gives values of steel stress at general bond slip for increasing values of embedment length of ¼, ⅜, and ½-in. prestress-

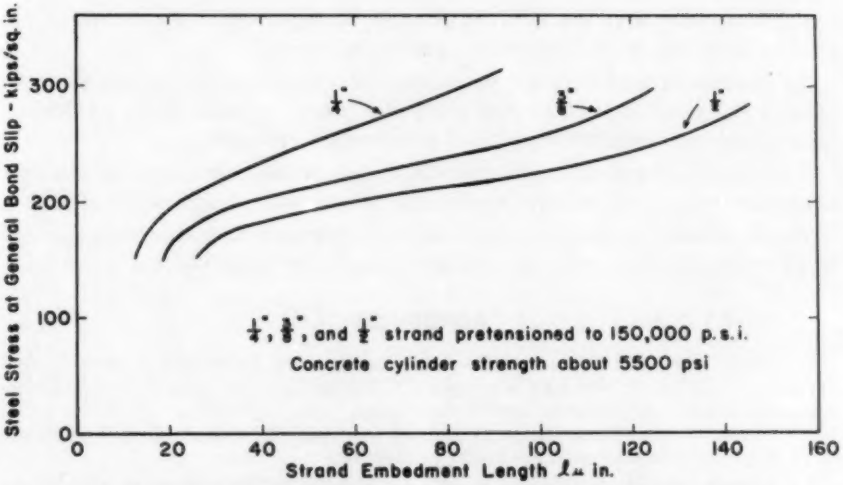


Fig. 6—Relation of steel stress at general bond slip to strand embedment length $l_u$

ing strand. Steel stresses for intermediate embedment lengths and strand sizes may be obtained with sufficient accuracy by use of linear interpolation between tabulated values.

## SUMMARY

The primary object of this investigation was to gain a fuller understanding of the flexural bond behavior of pretensioned prestressed concrete beams, and to develop design criteria. Particular attention was paid to the influence of strand size and embedment length on the bond performance of pretensioned beams. The influence of the following variables was also investigated to a limited extent: (a) percentage of steel reinforcement, (b) reduction of concrete strength, (c) strand surface condition, and (d) use of embedded end anchorages on pretensioned strand.

The results of tests on 47 beams support the flexural bond wave theory proposed by Janney,[1] and confirm that a general bond slip occurs in a pretensioned beam when the peak of the flexural bond stress wave reaches the stress transfer zone.

It was found that strand size and embedment length have a considerable influence on the value of the average bond stresses at which general bond slip occurs; this is clearly demonstrated in Fig. 5. The test results enabled curves to be drawn of bond stress at which slip will probably occur for particular strand sizes and embedment lengths. From these curves design criteria for the avoidance of general bond slip were obtained. These are in the form of curves and a table relating steel stress at which general bond slip will probably occur to embedment length for ¼, ⅜, and ½-in. prestressing strand.

Using these curves in design, it is possible to check quickly whether or not general bond slip is probable in any particular beam.

An increase in reinforcement percentage or a reduction in concrete strength reduces the possibility of general bond slip, since the steel stress at flexural failure, and the corresponding bond stresses, are reduced.

Rusting the strand raised the moment at general bond slip, and the ultimate moment of resistance, relative to identical beams with clean smooth strand.

The seven-wire strand develops additional beam strength, due to mechanical bond resistance, even after general bond slip to the beam end has occurred.

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## APPENDIX

### SHAPE OF FLEXURAL BOND STRESS WAVE IMMEDIATELY BEFORE GENERAL BOND SLIP

The following is an attempt to deduce the general form of the flexural bond stress wave for smooth, clean strand, using the test results reported in this paper.

It is necessary to make assumptions as follows:

1. The average transfer bond stress is 400 psi.
2. The bond wave shape immediately before general bond slip is a smooth curve with its peak at the end of the transfer zone.
3. At a given distance toward beam midspan from the end of the transfer zone, the local flexural bond stress will have a particular value regardless of the embedment length provided.

The value of 400 psi for average transfer bond stress is an average deduced from measurements on many pretensioned beams at the Portland Cement Association Laboratories. The average transfer lengths for strand pretensioned to 150,000 psi were found to be 13, 19, and 26 in. for ¼, ⅜, and ½-in. strand, respectively.

The average flexural bond stress between the end of the transfer zone and the section of maximum strand stress was calculated as follows. The area of the transfer zone bond stress block was subtracted from the product of the average bond stress $u_a$ and the embedment length $l_e$; the result was divided by the embedment length minus the transfer length. Hence

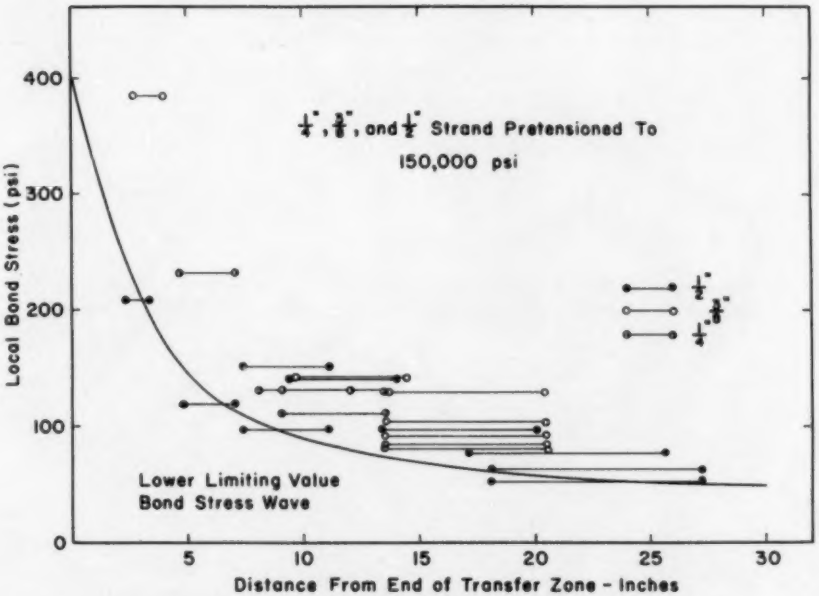


Fig. A-1—Relation of local bond stress to distance from end of transfer zone

the average bond stress between end of the transfer zone and a section of maximum strand stress may be expressed as follows:

$$u_a' = \frac{u_a l_u - 400 l_t}{(l_u - l_t)} \qquad \text{(A1)}$$

Values of $u_a'$ were calculated for various embedment lengths using the experimental values of $u_a$ obtained in this investigation.

Consider a smooth curve represented by the equation $y = F(x) = Ax^n = B$ where $x$ lies between zero and a finite value $x_1$, and where $n$ is between 1 and 4. It can be shown by algebra that for a given value of $n$ within this range, a horizontal line with ordinate equal to the average value of $y$ will cut the curve $y = F(x)$ at some point between $x = 0.50x_1$ and $x = 0.67x_1$.

It was thought that, for any particular embedment length considered, the flexural bond wave curve would fall between curves $u = Al + B$ and $u = Al^4 = B$, where $u$ = local bond stress at distance $l$ from the section of maximum strand stress, measured toward the free end of the strand. Values of $u_a'$ calculated using Eq. (A1) were therefore plotted in Fig. A-1 as horizontal lines extending from $l = 0.50\ (l_u - l_t)$ to $l = 0.67\ (l_u - l_t)$. The actual bond wave curve for each particular size strand should therefore at some point cut each of the horizontal lines which relate to the same strand size.

The experimental results for the different strand sizes overlap in Fig. A-1, and it is therefore reasonable to draw a single flexural bond wave curve for all strand sizes. The curve shown in Fig. A-1 was drawn so as to be a *lower limiting curve* for local flexural bond stress immediately prior to general bond slip.

Using this limiting curve of local bond stress it is possible to calculate the average bond stress for any particular strand size and embedment length as follows:

1. Measure the area under the curve of local bond stress over the embedment length considered, and add to this the area of the transfer bond stress block ($400l_t$).
2. Divide the area obtained by the total embedment length, and the result is the average bond stress over that embedment length immediately prior to general bond slip.

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Title No. 55-52

BOTH PLAIN AND AIR-ENTRAINED CONCRETES were made with aggregate of constant specific surface whose grading was varied while other mix proportions were held constant. Mixes made with a given aggregate grading which entrained more air under natural conditions (no air-entraining agent) also entrained proportionately more air with an air-entraining admixture. Relative effect of the different size fractions of aggregate on air entrainment are evaluated, assigning the greatest weight to material passing the No. 25 and retained on the No. 52 sieve. Reduction in sand content or increasing its fineness both tend to decrease air entrainment potential.

# aggregate grading affects air entrainment

B. G. SINGH

■ MANY DESIRABLE PROPERTIES are induced in fresh and hardened concrete by the entrainment of a controlled amount of air in the form of minute air bubbles. It is generally claimed that fresh concrete is more workable, bleeds and segregates less, and that the hardened concrete is more resistant to frost and sulfate action. Although air entrainment is influenced by several factors, it must be limited to obtain optimum benefits; thus the problem of control is important.

Among the factors of greatest practical importance for given temperature and atmospheric pressure and consistency are: type and concentration of the air-entraining agent; aggregate-cement ratio; and the grading of the aggregate. Work done in the past clearly shows that richer mixes require more of a given air entraining agent, to entrain the desired volume of air. As regards the grading of the aggregates, published results[1,2] indicate that as the sand becomes coarser, more air is entrained when all other factors are equal. In particular it is thought that entrained air increases with the amount of ma-

terial passing the No. 30 and retained on the No. 50 sieve. Though it is not disputed that the various size groups play some part in entraining air, indication of their relative importance is still somewhat lacking. This is the main subject of this report.

In the past, an important restriction in investigating the effect of grading more fully was the control of consistency with change in grading. However,

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this restriction has largely been overcome, for it has been shown[3] that consistency can be held reasonably constant for constant water-cement and aggregate-cement ratios, provided that the specific surface of the aggregate is constant. Advantage was taken of this fact in investigating the effect of grading on air entrainment.

Though the gradings used in this investigation are not those that are generally used in practice, they were, nevertheless, used to magnify the relative effect of each size group. The tests described cannot be regarded as exhaustive, though the conclusions drawn should be of sufficient practical importance to have some bearing on the assessment of an aggregate grading in relation to its air entrainment potential, which ultimately should lead to better control.

## METHOD OF INVESTIGATION

A single delivery of aggregate (classified as irregular by BS 882:1944) was screened into the various size groups, and gradings were made as required to give a constant specific surface. The cement used was blended and the same was used throughout the experiments. One air-entraining agent stirred into the mixing water in the ratio of 1:980 by volume was used for all mixes. The temperature and relative humidity at all experimental stages were constant. Proportioning of the mix was by weight.

The mixer used was a horizontal rotating type laboratory mixer. Ingredients were mixed 2 min before making the slump, compacting factor (CF),[4] and Vebe[5] tests* and the ASTM C 231 test for determining the volume of air for each mix. All these tests were performed simultaneously after mixing.

Nine aggregate gradings were made up with the separate size groups to give a specific surface of 32.0 sq cm per g. This rather high value was se-

*The CF test measures the relative compaction achieved by concrete falling under its own weight under standard conditions. CF values normally range between 0.70 and 1.00. A high value within this range represents a concrete of high workability.

The Vebe test result for practical purposes could be taken as the time required (in seconds) to vibrate a standard shape and volume of concrete to full compaction with the standard Vebe apparatus. A low value, as 1 Vebe degree, indicates a concrete that will flow readily under compactive effort.

lected to allow a wider variation in particle size distribution without introducing harshness with the fine sand and coarse aggregate mixes. These nine gradings (see Table 1) were used in 1:4½ mixes at a water-cement ratio of 0.45 and 1:6 mixes at a water-cement ratio of 0.55. Both of these mixes were made with and without air entraining agent.

## TEST RESULTS

Test results corresponding to each of the nine gradings are given in Table 1. It will be noted, after considering the variation of the slump, Vebe, and CF test results, that consistency (workability) was reasonably constant for the 1:4½ mixes without air-entraining agent (Series 1A) at the water-cement ratio of 0.45; similarly, consistency of the 1:6 mixes of Series 1B was constant at the water-cement ratio of 0.55. The consistency test values for Series 2A and 2B, i.e., 1:4½ and 1:6 mixes, respectively, with air-entraining agent, are also given in Table 1. The slump and the Vebe tests appear to be more sensitive to increases in workability with air entrainment, whereas the CF test for the 1:4½ mixes with the entrained air (Series 2A) was relatively insensitive. Though mixes with more air were more cohesive and resisted segregation to a greater extent, no significant increase in workability was measured for more air for given mix proportions.

The percentage volume of air given in columns 1A and 1B for 1:4½ and 1:6 mixes, respectively, is for naturally entrained air, that is, air entrained in the absence of an entraining agent under natural conditions. The percentage volume in columns 2A and 2B is for 1:4½ and 1:6 mixes, respectively, with air-entraining agent at the constant concentration of 1 part to 980 parts of water. As the latter values include both naturally and artifically entrained air, they will be, for convenience, referred to as gross entrained air.

Fig. 1 shows the relationship obtained between gross and naturally entrained air for all the mixes. The relationship seems quite definite and it may be concluded that the gross entrained air is related to the air entrained under natural conditions; that is, an aggregate that will entrain a large amount of air under natural conditions will entrain proportionately more air with an air entraining admixture.

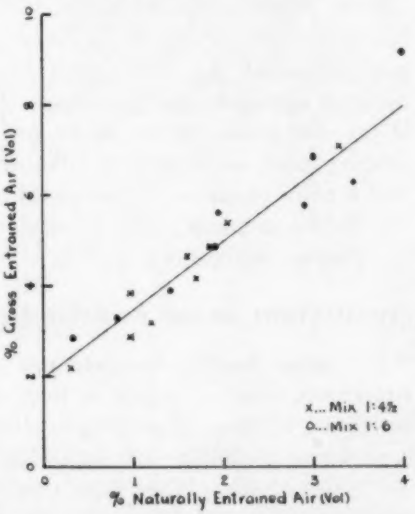


**Fig. 1—Relationship between gross and naturally entrained air. Concentration of air entraining admixture constant**

**TABLE 1—EFFECT OF AGGREGATE**

Specific surface of aggregate 32.0 sq cm per g. Air-entraining 1A and 1B denotes 1:4½ and 1:6 mixes without

| Grading No. | Aggregate grading, percent retained between given sieves* | | | | | | | 1:4½ mix, | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | No. 100 and No. 52 | No. 52 and No. 25 | No. 25 and No. 14 | No. 14 and No. 7 | No. 7 and 3/16 in. | 3/16 in. and 3/8 in. | ⅜ in. and 3/4 in. | Slump, cm | | Vebe degrees | |
| | | | | | | | | 1A | 2A | 1A | 2A |
| 1 | 25.4 | — | — | — | — | — | 74.6 | 1.0 | 4.5 | 10.7 | 4.5 |
| 2 | 15.4 | 15.4 | — | — | — | 34.6 | 34.6 | 2.5 | 4.5 | 9.0 | 4.5 |
| 3 | 12.9 | 12.9 | 12.9 | — | — | 30.6 | 30.7 | 1.5 | 4.5 | 8.0 | 4.5 |
| 4 | 11.7 | 11.7 | 11.7 | 11.7 | — | 26.6 | 26.6 | 1.0 | 3.5 | 8.0 | 4.5 |
| 5 | 11.2 | 11.2 | 11.2 | 11.2 | 11.2 | 22.0 | 22.0 | 1.0 | 3.5 | 7.8 | 4.0 |
| 6 | 24.0 | — | — | — | — | 38.0 | 38.0 | 1.75 | 2.3 | 6.0 | 7.5 |
| 7 | 18.5 | — | 9.2 | 9.2 | 9.2 | 26.9 | 27.0 | 1.0 | 3.0 | 11.0 | 5.0 |
| 8 | — | 35.9 | 7.2 | 7.2 | 7.2 | 21.2 | 21.3 | 0.0 | 1.8 | 7.9 | 6.5 |
| 9 | 17.1 | — | 17.1 | — | 17.1 | 24.3 | 24.4 | 1.7 | 2.5 | 8.0 | 5.0 |

*British standard sieve numbers.

Examination (see Table 1) of the amount of gross (or naturally) entrained air in relation to the amount of 25-52 size group* for each grading indicates wide scatter, though there is a general tendency for the percentage of air to increase with the amount of this size. A better relationship is, however, obtained (see Fig. 2) between the percentage of 52-100 size group and the amount of gross entrained air. As the amount of material of this size increases, there is a marked tendency for the air to decrease. This result could be explained in part as due to the fact that as more material of this size group was introduced, less of the other size groups had to be used to maintain a constant aggregate specific surface. It is, however, quite clear that particles of this size group are not so conducive to air entrainment. Although considering mixes made up with Gradings 6, 7, and 9, it should be equally clear that a required amount of air could be entrained in the complete absence of the 25-52 size group. The other sizes (including the 52-100 size) obviously also play an important part.

## EVALUATION OF THE RELATIVE EFFECT OF THE VARIOUS SIZE GROUPS

If a series of seven numbers, one corresponding to each of the aggregate size groups could be found so that when the percent of each size group is multiplied by its corresponding number and added for each grading, the sum in each case divided by 100 equals the percent of air entrained for any particular series of mixes, then these numbers will give a measure of the relative part played by each size group. The accuracy in the estimation of such constants is facilitated by the great variation in the nine gradings used.

*Approximates to the 30-50 size group of the previous investigators.

## GRADING ON AIR ENTRAINMENT

agent : water ratio = 1:980. Cement No. 499
agent, respectively, 2A and 2B with agent

| $W/C$ = 0.45 | | | | 1:6 mix, $W/C$ = 0.55 | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| CF | | Air content, percent by volume | | Slump, cm | | Vebe degrees | | CF | | Air content, percent by volume | |
| 1A | 2A | 1A | 2A | 1B | 2B | 1B | 2B | 1B | 2B | 1B | 2B |
| 0.87 | 0.88 | 0.30 | 2.25 | 0.5 | 2.6 | 15.0 | 13.0 | 0.85 | 0.88 | 0.3 | 2.9 |
| 0.84 | 0.88 | 0.95 | 3.9 | 0.5 | 3.7 | 12.0 | 5.0 | 0.84 | 0.88 | 1.9 | 4.95 |
| 0.85 | 0.88 | 1.60 | 4.7 | 0.5 | 3.0 | 10.0 | 5.5 | 0.85 | 0.87 | 2.9 | 5.8 |
| 0.85 | 0.87 | 1.85 | 4.9 | 0.5 | 3.5 | 11.5 | 4.5 | 0.85 | 0.89 | 3.0 | 6.95 |
| 0.87 | 0.88 | 2.05 | 5.4 | 0.5 | 3.0 | 11.8 | 5.0 | 0.85 | 0.89 | 3.45 | 6.3 |
| 0.85 | 0.87 | 0.95 | 2.9 | 0.5 | 3.5 | 9.3 | 5.0 | 0.84 | 0.88 | 0.8 | 3.35 |
| 0.84 | 0.88 | 1.20 | 3.2 | 0.5 | 2.5 | 8.7 | 5.5 | 0.85 | 0.89 | 1.4 | 3.95 |
| 0.87 | 0.87 | 3.30 | 7.15 | 0.3 | 1.5 | 15.7 | 9.0 | 0.84 | 0.87 | 4.0 | 9.25 |
| 0.86 | 0.87 | 1.70 | 4.20 | 0.5 | 3.0 | 9.3 | 4.5 | 0.83 | 0.90 | 1.95 | 5.65 |

The seven numbers were selected by a process of trial and error so that the result gave the best agreement with the experimental values. The nearest whole numbers obtained for the conditions of the 1:6 mixes with the gross entrained air, are:

| Size group | 52-100 | 25-52 | 14-25 | 7-14 | 3/16-7 | 3/8-3/16 | 3/8-3/4 |
|---|---|---|---|---|---|---|---|
| Constant | 8 | 20 | 15 | 8 | 4 | 2 | 1 |

It was found that when seven numbers for the 1:4½ mixes were determined similarly, they were for practical purposes related to the above in the ratio of 4½:6. Therefore, the values estimated for 1:6 mixes were multiplied by 0.75. All the values so estimated are plotted against the experimental values in Fig. 3. A coefficient of correlation of 0.96 was obtained between the ex-

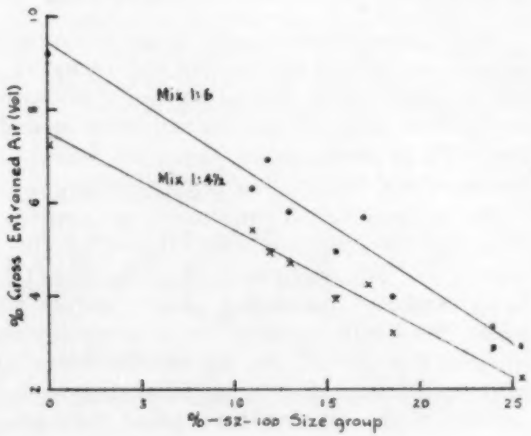


Fig. 2—Gross entrained air plotted against percentage of 52-100 size group. Aggregate of constant specific surface, 32 sq cm per g

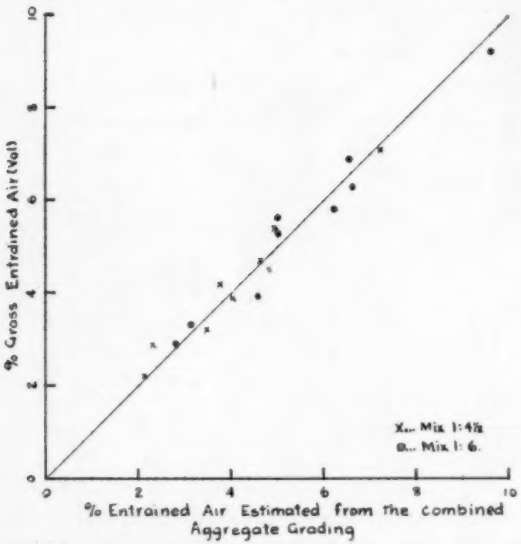


**Fig. 3—Relationship between gross entrained air and that estimated from coefficients for combined aggregate grading**

perimental and estimated. More exhaustive methods to select better numbers were not used because of the limited data.

Although the numbers are relative and could not be applied directly, they should find practical use in evaluating a sand, coarse aggregate, and any combination of these in relation to their air entraining properties. A low value will indicate a low potentiality and the converse would be true. An aggregate combination, desirable for other reasons, giving a low value will indicate a higher concentration of air-entraining agent. The value calculated as indicated might be called the modulus of air entrainment.

## DISCUSSION

As the gross entrained air is proportional to the naturally entrained air, it seems reasonable to assume that the mechanics of entrainment in both cases are the same, except that in the case of the former, more air is entrained and the bubbles are more uniform and better spaced. It could also be assumed that with an air-entraining agent, air present, or that which enters the mix during mixing, is more readily stabilized.

The mechanics of air entrainment are complex and difficult to explain fully. However, the following theory based on the author's observation, seems reasonable. Fine particles (such as the 52-100 size group) have smaller interfacial capillaries (producing greater surface tension) and these are more readily filled with water, cohere more readily and so resist shearing and are rendered less mobile. As the interfacial capillaries are small and are filled more readily soon after the addition of water to the mix, less air will be surrounding them. On the other extreme, the coarse particles become surrounded

by the finer particles and sand-cement matrix and as a result are rendered almost inert in relation to air entrainment. However, the particles of the intermediate size, due to larger capillaries (lower surface tension), will cohere less, contain more air and water in a relative free condition. Thus the particles of the intermediate size groups satisfy to a greater extent, the physical conditions necessary for air entrainment.

The particles of the intermediate size groups will also permit the entry of air into the mix, during mixing, more readily and facilitate effective shearing of the mix and breaking up the air present (or that which enters) into minute bubbles suitable for stabilizing. The author believes that the constant assigned to each size group is a relative measure of their total effect in relation to these factors.

Although particles passing the No. 100 sieve were not investigated, extrapolation from the present results indicate that their effect will most probably be negligible under practical conditions. However, the effect of particle shape and surface texture might be of importance in the light of the foregoing discussion. Two aggregates of the same particle size distribution may not have the same air entraining potential if the particle shape and surface texture of each are different. An investigation of the effect of shape and surface texture, especially of more angular particles, should be of interest, as it is with these that concrete offers greater resistance to compaction and therefore is likely to derive greater benefit from purposeful air entrainment.

Gradings with all particle size groups are normally required by specifications. Such gradings entrain more air under natural conditions. Such traditional gradings also produce durable concrete though the importance of the entrained air under such conditions is a point of conjecture. If this naturally entrained air induces some desirable weathering properties, especially in relation to frost, then it should be expected that aggregates that entrain less air will produce less durable concrete. Engineers may find it economical to use finer sands and this may produce concrete with less air. Concrete of comparable strength[6] could be proportioned in a rational manner with such sands though it is likely to suffer greater segregation. Use of air-entraining agent to reduce segregation and adjust air content may well be a factor determining whether such materials are used for better concrete when available at much lower cost.

## ACKNOWLEDGMENTS

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# CONCRETE BRIEFS

## Notes from Field and Office

Re-Examination of Nichols' Expression for the Static Moment in a Flat Slab Floor... 811
Problems and Practices ..... 814

## Re-Examination of Nichols' Expression for the Static Moment in a Flat Slab Floor

By C. P. SIESS*

### SLABS WITH CIRCULAR CAPITALS

The original expression derived by Nichols† for square panels and circular column capitals can be expressed in the following form:

$$M_o = \frac{WL}{8}\left[1 - \frac{4}{\pi}\frac{c}{L} + \tfrac{1}{3}\left(\frac{c}{L}\right)^3\right] \quad \ldots\ldots (1)$$

where $M_o$ = sum of the total positive moment at midspan across the full width of the panel and the total negative moment on the column lines and around the periphery of the column capital
$W$ = total uniformly distributed load on the panel
$L$ = span length, center to center of columns
$c$ = diameter of column capital

For rectangular panels, the following expression can be derived in the same way as Eq. (1), although this was not done explicitly by Nichols:

$$M_o = \frac{WL_1}{8}\left[1 - \frac{4}{\pi}\frac{c}{L_1} + \tfrac{1}{3}\left(\frac{c}{L_1}\right)^3\frac{L_1}{L_2}\right] \quad \ldots\ldots (2)$$

where $L_1$ = span in direction of moments considered and $L_2$ = span in other direction.

Both of these expressions are based on the same set of assumptions, namely:

(a) The floor consists of a large number of identical panels extending indefinitely in both directions. For practical purposes, this condition is satisfied for interior panels in an array consisting of a relatively large number of identical panels extending in both directions.

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†Nichols, John R., "Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors," *Transactions*, ASCE, V. 77, 1914, pp. 1670-1681.

(b) All panels are uniformly loaded with the same intensity of load.

(c) The shear (reaction) at the edge of the column capital is uniformly distributed around the circumference of the capital.

In the closing discussion of his paper, Nichols* suggested an approximate expression as an equivalent to Eq. (1), as follows:

$$M_o = \frac{WL}{8}\left(1 - \tfrac{2}{3}\frac{c}{L}\right)^2 \quad \ldots\ldots (3)$$

He stated that the error involved in using Eq. (3) instead of Eq. (1) was less than 1 percent for values of $c/L$ less than 0.3. The error may actually be slightly greater than this, being about 2 percent for square panels, about 3 percent for a rectangular panel with $L_1 = 0.5\ L_2$ (short span), and less than 1 percent for $L_1 = 2\ L_2$ (long span). In all cases Eq. (3) is on the conservative side.

Since Eq. (3) is somewhat simpler than Eq. (1) or (2) and is clearly almost as accurate, it was adopted for use in the specifications and now appears in Section 1004(f) of "Building Code Requirements for Reinforced Concrete (ACI 318-56)." In this case, however, only the correction term for $c/L$ remains intact since the coefficient 0.125 for static moment has been reduced to $0.09\ F$, yielding values ranging from 20 to 28 percent less than Eq. (1) or (2).

Although Eq. (3) is both simple and accurate, an even simpler and more accurate approximation to Eq. (1) and (2) can be written as follows:

$$M_o = \frac{WL_1}{8}\left(1 - 1.25\frac{c}{L_1}\right) \quad \ldots\ldots (4)$$

For a value of $c/L = 0.3$, Eq. (4) is in error by only about 0.2 percent on the low side as compared to Eq. (1) for square panels. For rectangular panels, and for $c/L_1 = 0.3$, Eq. (4) is 0.5 percent high for $L_1 = 0.5\ L_2$ and 1.7 percent low for $L_1 = 2\ L_2$.

The choice between the form of Eq. (3) and that of Eq. (4) for use in the ACI Code is of course a matter of opinion. The difference in accuracy is trivial in view of the 20 to 28 percent reduction from the true static moment already provided in the Code. The difference in complexity may also be of little significance. Moreover, since both equations are only empirical approximations to the original rational expression, the derivation of which is either not known or has been forgotten by most engineers, there is little basis for a choice in terms of tradition. Nevertheless, it is suggested that the form of Eq. (4) be adopted in the Code on the basis of the principle that an approximate calculation should be made in the simplest manner possible.

## SLABS WITH RECTANGULAR CAPITALS

Since the flat plate type of construction usually involves the use of square or rectangular columns, the "capital" in this case is no longer circular but of the same shape as the column. Expressions for the static moment in such floors can easily be derived by the same method as that used by Nichols, and are offered here as being of possible interest.

Two cases are considered. In both, the assumptions previously listed as (a) and (b) are made just as they were by Nichols. However, two different assumptions are made regarding the distribution of shear around the periphery of the capital: (1) that the shear is uniformly distributed, and (2) that all of the shear is concentrated at the corners of the column or capital.

*Ibid., p. 1735.

### Uniformly distributed shear

The general expression for total or static moment is

$$M_o = \frac{WL_1}{8}\left[1 - \frac{2 + \frac{c_1}{c_2}}{1 + \frac{c_1}{c_2}}\frac{c_1}{L_1} + \frac{1}{1 + \frac{c_1}{c_2}}\frac{{c_1}^2 c_2}{{L_1}^2 L_2}\right] \quad \ldots\ldots(5)$$

where the notation is the same as for Eq. (1) and (2) except that $c_1$, and $c_2$ = over-all dimensions of rectangular capital in directions of spans $L_1$ and $L_2$, respectively.

For square capitals, Eq. (5) reduces to

$$M_o = \frac{WL_1}{8}\left[1 - \frac{3}{2}\frac{c}{L_1} + \frac{1}{2}\left(\frac{c}{L_1}\right)^2 \frac{L_1}{L_2}\right] \quad \ldots\ldots(6)$$

An approximation to Eq. (6), in the same form as Eq. (4), is:

$$M_o = \frac{WL_1}{8}\left(1 - 1.45\frac{c}{L_1}\right) \quad \ldots\ldots(7)$$

### Concentrated shear

The general expression for static moment is

$$M_o = \frac{WL_1}{8}\left[1 - 2\frac{c_1}{L_1} + \frac{{c_1}^2 c_2}{{L^2}_1 L_2}\right] \quad \ldots\ldots(8)$$

For square capitals

$$M_o = \frac{WL_1}{8}\left[1 - 2\frac{c}{L_1} + \left(\frac{c}{L_1}\right)^2 \frac{L_1}{L_2}\right] \quad \ldots\ldots(9)$$

An approximation to Eq. (9), good to about $c/L = 0.25$, is

$$M_o = \frac{WL_1}{8}\left(1 - 1.95\frac{c}{L_1}\right) \quad \ldots\ldots(10)$$

Comparison of Eq. (7) and (10) for square capitals with Eq. (4) for circular capitals indicates that the use of Eq. (10) in Section 1004(f) of the ACI Code will yield conservative results for square or rectangular capitals if $c$ is taken as the over-all dimension. This, however, is of little importance in view of the large reduction from the static moment that has been adopted in the Code. Moreover, uncertainties regarding the actual behavior of slabs with square capitals is sufficient justification for some degree of conservatism even in addition to that provided by the coefficient $F$ for small capitals which will apply more frequently to square than to round capitals.

# Problems and Practices

*A series relating to "down-to-earth, everyday" concrete problems which attempts to give brief answers to the more common (and sometimes uncommon, too) questions asked about concrete design and construction practices.*

*To some, the answers will seem simple and obvious; others may prove controversial.*

*All ACI members are invited to participate—either by submitting an inquiry, or even better, by telling JOURNAL readers how an intriguing problem was solved. It may well be that readers will be able to suggest more practical solutions than those presented.*

---

*Q. Bending moments in flat slab structures (without drop panels and without capitals), when total depth of the spandrel beams at exterior supports is equal to the slab thickness, are determined by elastic analysis on the basis of rigid connections between columns and slabs; i.e., the stiffness of the columns is taken into account. Design of such structures is treated in Section 1003 of "Building Code Requirements for Reinforced Concrete (ACI 318-56)." The Code is quite clear in respect to design of the slabs and columns but does not include recommendations for design of the joints between columns and slabs, except a statement that these oints may be considered rigid.*

*My question, therefore, refers specifically to the design of the joints between columns and slabs, which have to transfer the computed bending moments from the slabs into the columns. To be more specific, I shall limit the question to the joints between the exterior columns and the slabs, as illustrated in Fig. 1. The end bending moment for the whole bent (120 ft-kips in Fig. 1) is transferred through the three faces in the slab (through face* bc, *as a bending moment; through faces* ab *and* cd, *as torque).*

*The quantity of reinforcement which is usually placed over the exterior column, varies from 25 to 50 percent of that required for the end bending moment. This quantity of reinforcement is within ACI Code requirements. The designed resistance to bending of Face* bc *varies within the same limits, i.e., Face* bc *can transfer a bending moment of about 25 to 50 percent of the end bending moment. (By "designed resistance" is implied a resisting bending moment computed on the basis of ACI allowable stresses, or a plastic bending moment with a factor of safety of about 1.8.) It follows that Faces* ab *and* cd *should transfer by torsion the remaining 50 to 75 percent of the end bending moment.*

*In structures of the type described, and commonly encountered in practice, the computed St. Venant's torsional (shearing) stress on Faces* ab *and* cd *may be as high as 0.7* $f_c'$, *is seldom below 0.2* $f_c'$, *and is usually about 0.3* $f_c'$. *Shearing stresses from the vertical shearing force (reaction) may increase the maximum shearing stress by 10 to 20 percent. However, the magnitude of the latter is, for the most part, a fraction of the torsional shearing stress.*

*1. Implicity, the ACI Code does not require any precautionary design measures for the joints between columns and slabs, except the provisions of reinforcement in the column strip, and certain limitations on the size of columns with respect to their stiffness. Many structures have been designed on this basis, and although the computed torsional stresses in the joints of these structures are quite high, their performance is satisfactory. Are the mentioned ACI provisions applicable, irrespective of the size of the columns (except the stiffness requirements), and should the torsional stresses be disregarded?*

*2. If every structure should be judged on its merits, and if the joints between columns and slabs should be given due consideration in the design, how should the end bending moment (such as the 120-ft-kips in Fig. 1) be proportioned among Faces* bc, ab, *and* cd?

*3. What is the permissible torsional (shearing) stress on Faces* ab *and* cd.

*a. Without provision of stirrups or longitudinal bars in the spandrel (except those required for vertical shear and*

*for bending moment in the longitudinal column strip)?*

*b. With provision of additional stirrups and longitudinal bars in the spandrel to resist tensile components of the torsional stresses?*

*Although Faces* ab, bc, *and* cd *have to transfer the end bending moment, we realize that the resistance of these faces cannot be computed as the sum of the usual bending resistance of Face* bc, *and St. Venant's torsional resistance of Faces* ab *and* cd. *We believe that the following factors would increase the resistance of such a joint*: (*a*) *warping torsional stresses set up at Faces* ab *and* cd *and* (*b*) *confinement of concrete in the slabs around the columns. Investigation carried out in our office indicates that the contribution of warping stresses is not of a major nature, principally because the rotations of the spandrel are relatively small.*

*Effect of the confinement of the concrete has also been studied by us on the basis of Mohr's rupture envelope as well as St. Venant's strain failure theory. Because of the limitations in pursuing a study of this nature in a consulting engineering office, this study has been only superficial. However, a relatively rough investigation indicated to us that the capacity of the joint designed by the ACI Code does not provide sufficient safety for the full end moment.*

LEV ZETLIN
Consulting Engineer
New York, N. Y.

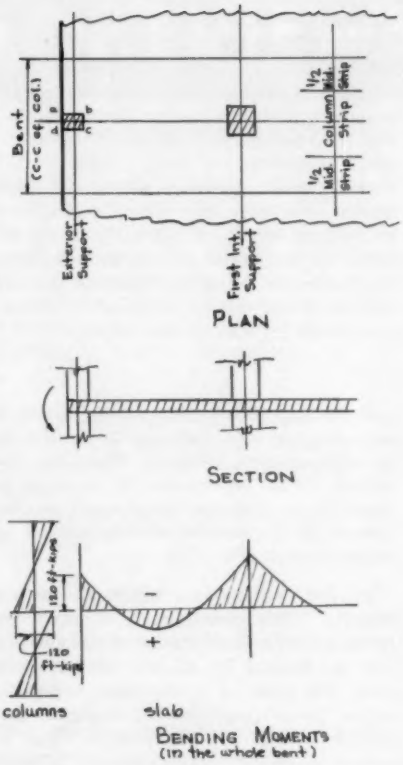


Fig. 1

*A.* This question is a complex one for which no simple answer is possible. Obviously practicing designers have solved the problem in various ways. Perhaps the publication of the question will develop some of these answers.

The philosophy of the approach to this type of problem is best explained by the former chairman of Committee 318, Frank Kerekes, in "Review of Changes in the ACI Building Code Requirements for Reinforced Concrete."* "The experienced practical designer should take into account as many of the forces and influences as he can determine for the particular structure he has under consideration. His analysis of especially unusual structural types and systems should anticipate every possible elastic and geometric action. Then he can, to good advantage, be guided by appropriate and related provisions of the Code. But the provisions of a building code are not intended to make up for lack in theoretical understanding, for the judgment to benefit by the experiences of others, and for the skill in anticipating the behavior of related portions of a complex structural system. Once the action of each component part of a complex structure is analyzed, and the interaction of the component parts is determined, the building code provisions can be utilized to good advantage and with a reasonable amount of assurance."

Section 1003 on elastic analysis assumes by implication details structurally adequate for the resulting design. Such details are left to the designer's judgment since he is pro-

*ACI JOURNAL, V. 29, No. 3, Sept. 1957 (*Proceedings*) V. 54), pp. 185-196.

ceeding through his own analysis. Under Section 1004 on empirical design an external column moment of $WL/30$ is prescribed. If provision for transferring full moment to the column through bending is neglected approximately 75 percent may remain for transfer by torque in an empirical design. Since the ACI Building Code presumes all tensile stress carried by reinforcement except that allowed in diagonal tension resulting from vertical shear, by implication a designer is expected to provide additional reinforcement and not depend on concrete unreinforced for torsion to transmit moment to the column.

* * *

*Q. Is unprotected concrete pipe suitable to carry chemical waste including dilute solutions of sodium maleate, phthalate, thiosulfate and sulfate? What information is available on resistance of concrete to these agents or what methods of test could be used to evaluate its resistance?*

*A.* The sulfates are aggressive toward concrete. This is true even of very weak solutions. The U. S. Bureau of Reclamation *Concrete Manual* (p. 12, 6th edition, 1955) gives 150 ppm as a threshold value for sulfate (as $SO_4$) as between negligible and positive attack. It recommends Type II cement when the concentration is between 150 and 1000 ppm and Type V for higher concentrations.

No information has been located on sodium maleate and phthalate or other maleates or phthalates. Though they evidently give basic solutions and would not be objectionable from that standpoint, it is entirely possible, and there is even some reason to suspect, that they might perhaps have undesirable specific reactions. It would be advisable to test them.

Test procedures are given in "Standard Method of Test for Chemical Resistance of Hydraulic Cement Mortars," ASTM C 267-54 *1955 Book of ASTM Standards*, Part 3). This test could be accelerated by scheduling brushings at frequent intervals, by circulating the solution, by elevated temperature, or by otherwise modifying the procedure.

Thiosulfates are regarded as having somewhat the same effects as sulfates owing to tendency to decompose and produce some of the latter. One source of reference is A. Kleinlogel's book on *Influences on Concrete* (English translation of 1941 German edition published by Frederick Ungar Publishing Co., New York, 1950). Kleinlogel states regarding sulfur and sulfur compounds that they may, in any form, have either directly or indirectly harmful effects on concrete. He adds, "Above all, it is always the sulfuric acid or the sulfates which are formed through oxidation of other sulfur compounds or are already present as sulfate which causes destruction." Latimer and Hildebrand's *Reference Book of Inorganic Chemistry* attributes decomposition of sodium thiosulfate solution to the carbon dioxide in the air, and presumably protection against $CO_2$ might greatly diminish it. However, it is possible that thiosulfate may itself have a specific effect somewhat like that of sulfate. The destructive effect of sulfate is due to formation of a calcium aluminate sulfate (commonly called calcium sulfoaluminate) and thiosulfate itself can form a complex with aluminate that may be sufficiently similar to the destructive calcium aluminate sulfate. Presumably if testing with the maleate and phthalate it would be desirable to test the thiosulfate and sulfate at the same time.

Another general source of reference is E. Rabald's *Corrosion Guide* (Elsevier Publishing Co., New York, 1951) which reports the effects of chemicals on concrete as well as metals and alloys. It lists sodium thiosulfate but leaves blank the space allotted to concrete, thus indicating that nothing was found regarding the effect of this material.

# CURRENT REVIEWS

**of Significant Contributions in Foreign and Domestic Publications**

## Bridges

### Composite construction of bridges using steel and concrete

R. David and G. G. Meyerhof, *The Engineering Journal* (Montreal), V. 41, No. 5, May 1957, pp. 41-47
Reviewed by Aron L. Mirsky

A summary, brief but thorough, of composite construction, including prestressed types. Outlines methods of analysis of dead and live load stresses, failure loads, effects of shrinkage, creep, and temperature, and demonstrates that the effect of different coefficients of thermal expansion of steel and concrete may be important, particularly in continuous structures exposed to the elements. A simply-supported composite beam is analyzed as a worked example, in the appendix.

### Composite steel and reinforced concrete centering for a bridge at Revin (in French)

B. Pierre, *Travaux* (Paris), No. 285, July 1958, pp. 543-554
Author's Summary

Having regard to the fact that concrete can transmit compressive forces at an appreciably lower cost than timber or steel, the author constructed self-supporting centering for building a bridge over the River Meuse. This centering comprised: (1) a light steel arch framework; and (2) a reinforced concrete slab placed over the extrados of this framework and connected to it.

When the centering was subjected to its working load, the reinforced concrete slab absorbed the direct compressive forces, while its smooth-finished upper surface served as formwork to the soffit of the arch of the bridge. The steel framework, which initially carried the slab during its construction, subsequently functioned as a stiffening girder and took up the bending to which the centering was subjected in consequence of the unequal loads acting upon it during the construction of the bridge arches. Article describes the technical solutions adopted, the design methods employed, and the procedure of operations.

### History of highway bridges in the province of Quebec (Historique des ponts-routes dans la province de Québec)

Louis-Philippe Gravel, *L'Ingénieur* (Montreal), V. 43, No. 170, Summer 1957, pp. 11-17; No. 171, Autumn 1957, pp. 31-36
Reviewed by Aron L. Mirsky

Fascinating history, well illustrated. The bridges of Quebec, spanning from 1789 to 1957 in time, form a history in miniature of bridge engineering.

### The Abidjan railroad-highway bridge (Le pont rail-route d'Abidjan)

Nicolas Esquillan, *Mémoires*, Société des Ingénieurs Civils de France (Paris), V. 111, No. 2, Mar.-Apr. 1958, pp. 97-118
Reviewed by Aron L. Mirsky

The old floating crossing over Ebrié Lagoon in Abidjan on the Ivory Coast (French West Africa) having become inadequate, a new crossing was planned. The Administration proposed a two-level steel truss structure. The structure as constructed, dictated largely by poor foundations, consists of twin box girders of precast prestressed concrete, 41 m in span, carrying highway vehicles on the upper deck and railroad trains inside the boxes, with cantilevered sidewalks and bicycle paths. All components—piers, gird-

A part of copyrighted Journal of the American Concrete Institute, V. 30, No. 7, Jan. 1959, *Proceedings* V. 55. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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ers, etc.—for the 8 main (lagoon) spans and the approach viaducts were precast on the bank and floated into position. Pozzolanic blast furnace cement was used to prevent damage by aggressive fumes or organic material in the mud.

The project is noteworthy, not only for its structural concept and solution, but for an elaborate system of approaches, complete with interchanges, plazas, and three-level grade-separation structure, which would do credit to Los Angeles, to say nothing of Darkest Africa.

### Bridge construction (Brückenbau)

Fritz Leonhardt, *VDI Zeitschrift* (Düsseldorf), V. 100, No. 18, June 21, 1958, pp. 789-798
Reviewed by Aron L. Mirsky

Report of developments since the last periodic review of the field (title source, V. 98, No. 17, June 11, 1956, pp. 944-952; "Current Reviews," ACI Journal, Feb. 1957, p. 811). Brief interest-whetting mentions are made of some very interesting recent structures.

### Masonry arch bridges: a study

Cyril S. Shettoe and William Henderson, *Proceedings*, Institution of Civil Engineers (London), V. 7 (session 1956-57), August 1957, pp. 723-762; discussion, p. 762-774*
Reviewed by Aron L. Mirsky

Results of field tests of 14 masonry arch bridges of various types indicate that masonry arches in sound condition behave elastically; that, with normal filling, composite action need not be taken into account to explain their behavior; and that the usual assumption of 45 deg dispension of live load through the fill is satisfactory in estimating the effective width and length of load on the arch. It was also found that abutment movement occurred in every bridge tested. Based on these results, authors suggest that elastic analysis be used, with limits imposed on the permissible stresses and eccentricity of thrust, and that an empirical allowance be made for abutment movement.

Reviewer would add that it is unfortunate that the old voissoir arch is a vanishing breed, in this day of the stone-faced concrete arch/frame.

*Additional discussion is printed in the issue of Dec. 1957, pp. 432-434. The dead-load trailer used in testing the bridges was described in *Engineering*, V. 173, No. 4506, June 6, 1952, pp. 721-722 ("Current Reviews," ACI Journal, Dec. 1952, p. 343.)

## Construction

### Designing an underground building

Ralph S. Torgerson, *Consulting Engineer*, V. 10, No. 4, Apr. 1958, pp. 92-95
Reviewed by Aron L. Mirsky

Brief description of underground civil defense emergency operations center at Portland, Oregon. Structural, mechanical, electrical problems and solutions involved are outlined.

### Modern slip- and climbing forms (Neuzeitliche Gleit- und Kletterschalungen)

Gotthard Franz, *VDI Zeitschrift* (Düsseldorf), V. 100, No. 3, Jan. 21, 1958, pp. 81-92
Reviewed by Aron L. Mirsky

A well-illustrated survey of the various types and recent applications of slipforms and climbing forms.

### Large "shell" dome

*Concrete and Constructional Engineering* (London), V. 53, No. 9, Sept. 1958, pp. 313-319

A doubly curved dome, 125 ft square, has been built to cover two tennis courts at Wimbledon, England. Boundary members of the shell are supported on four corner columns. The shell is 3 in. thick except for a width of 11 ft 6 in. along each side in which the thickness is increased, and has the form of a paraboloid with a span of 175 ft and a rise of 24 ft 4¾ in. Design and construction details are given.

### Construction of the exposition palace of the Centre National des Industries et Techniques (in French)

M. A. Balency-Bearn and M. G. Lacombe, Supplement to *Annales de L'Institut Technique du Batiment te des Travaux Publics* (Paris), No. 127-128, July-Aug. 1958, Series Technique Général de la Construction (24), pp. 876-902

The Exposition Palace has the form of a three-faced ribbed vault with a double shell. The total span of the rafter is 820 ft for a rise of 152 ft and the span of the tympanums is 715 ft between abutments. The thickness of the double shell is 5 ft 11 in. at the key of the vault and the total surface covered is 336,800 sq ft. The thrusts of the vault on the three abutments are absorbed by hard steel tie rods located in the façades.

Three different types of floor were used. A double slab technical floor of 115,738 sq ft constructed at the center of the building at a

height of 39 ft 5 in. on a triangular plan is supported by 51 columns; it consists of prefabricated prestressed elements.

Four floors totaling 570,487 sq ft on four levels run along the sides of the triangle covered by the vault. They consist of ribbed slabs resting on prefabricated beams. Lastly, 33,291 sq ft of standard floors on two levels are located at the corners of the building.

Double circulation stairs run to a height of 59 ft without intermediate supports.

### A decade of progress in reinforced concrete construction in Germany, (Zehn Jahre Entwicklung des Stahlbetonbaues in Deutschland)

WALTER NAKONZ, *Der Bauingenieur* (Berlin), V. 33 No. 1, Jan. 1958, pp. 2-6

Reviewed by ARON L. MIRSKY

An interesting review of progress in German concrete construction practices over the past decade, ranging from mechanization to precasting and prestressing, and placing considerable emphasis on the increase in specified strength of structural concrete.

### Wuppertal municipal pool (Das Stadtbad Wuppertal)

*Der Bauingenieur* (Berlin), V. 32, No. 9, Sept. 1957 pp. 344-359

#### Planning, layout, construction, costs

FRIEDRICH HETZELT, p. 344-348

#### Design of a suspended roof of lightweight concrete and technical considerations

FRITZ LEONHARDT and WOLFHART ANDRA, pp. 349-353

#### Layout and construction of the prestressed suspended roof

H. J. EULITZ, pp. 353-359

Reviewed by ARON L. MIRSKY

Main structure contains swimming and diving pool and grandstand-type seats; attached building has dressing rooms, etc., and a shallow pool for nonswimmers. Former overshadows latter, both for its spectacular external appearance and its construction: east and west walls are inclined (to accommodate the seats for spectators) with the upper portions extending beyond the vertical wall-supports; the singly curved suspended roof, of lightweight prestressed concrete, spans 65 m (chord length) between these walls. Reasons for single curvature, rather than double curvature as in the Livestock Judging Pavilion at Raleigh, N. C. (*Engineering News-Record*, Feb. 5, 1953, pp. 31-37; "Current Reviews," ACI JOURNAL, June 1953, p. 975) are discussed by Leonhardt and Andrä. Design of roof as a cylindrical-type shell was checked by test-loading.

### Portishead "B" power station, with particular reference to the circulating-water works

HORACE D. MORGAN, CHARLES K. HASWELL, and LOUIS SANCHA, *Proceedings*, Institution of Civil Engineers (London), V. 7 (session 1956-57), August 1957, 775-816; discussion, pp. 816-826*

Reviewed by ARON L. MIRSKY

An interesting paper describing construction of a large (360 million watt ultimate capacity) power plant on the Severn Estuary. Complicating factors included a 51 ft maximum tidal range, complicated geological formations (including coal strata), and sloping bedrock. In the latter stages of construction, fly ash from the first boilers to be commissioned was used in the concrete.

*For additional discussion, see issue of Dec. 1957, pp. 435-436.

## Construction Techniques

### Structures with self-supporting reinforcement

M. MENCL, *Concrete and Constructional Engineering* (London), V. 53, No. 10, Oct. 1958, pp. 349-355

Describes the erection of monolithic reinforced concrete structures in Czechoslovakia with welded self-supporting frames of reinforcement to which steel netting with a mesh of about 1/6 in. square is attached. Wires of the netting are at 1/5-in. centers and have a diameter of about 1 mm; weight is about 0.41 lb per sq ft. System avoids use of almost all formwork and centering. In placing the concrete it is said that only a negligible quantity of concrete falls through the mesh of the netting.

### Lamps heat concrete to keep work moving at Noxon Rapids Dam

RALPH MONSON, *Contractors and Engineers*, V. 55, No. 10, Oct. 1958, pp. 28-35

Radiant heat from banks of infrared lamps warmed certain structural concrete sections of the Noxon Rapids Hydroelectric Develop-

ment, Montana, during cold weather construction. Contractor used this method of heating horizontal construction joints in structural concrete. These assemblies were suspended about 10 ft above the construction joints of the concrete. Before a placement was started, the lights were turned on for a short period to melt any ice in the forms and to heat form and concrete surfaces. The heat lamps were kept on while concrete was being placed and for as long as the cold period lasted or until curing was complete. The surface being heated was covered with a thin layer of water to keep the surface from drying. Forms were left in place to provide insulation for vertical concrete surfaces. Water and aggregates were heated when necessary and the concrete was usually placed at 45 to 55 F.

## Design

### Analysis of membrane stresses in a thin shell of revolution (Calcul des contraintes de membrane dans un voile mince de révolution)

P. Laybangues, *Annales des Ponts et Chaussées* (Paris), V. 127, No. 5, Sept.-Oct. 1957, pp. 517-547
Reviewed by Aron L. Mirsky

Starting with the general equations of equilibrium, author obtains simple expressions which are then used to derive the equations of equilibrium of a tank or tower whose median surface is a hyperboloid of revolution of one sheet, under any distributed-type loading normal to the surface. In particular, author then investigates stresses due to wind in hyperbolic and truncated-cone towers and elliptic domes.

### Analysis of continuous cylindrical shells by the method of successive approximations

Gunhard Oravas, *The Structural Engineer* (London), V. 36, No. 8, Aug. 1958, pp. 253-266
Author's Summary

A method of successive approximations is presented for the analysis of reinforced concrete cylindrical shells of rotational symmetry continuous at their boundaries and of constant thickness.

This problem is of practical importance in the design of tanks, bins, and pressure vessels. This method is based on simplifying assumptions associated with reinforced concrete and therefore subject to definite limitations in its applications. A set of design graphs included aids the designer's numerical work. Two simple applications of the method are given.

### Combined footings on elastic soil (in Hebrew)

Z. Getzler, *In the Field of Building, Bulletin* No. 55, Technion Israel Institute of Technology (Haifa), 1958, 16 pp.
Author's Summary

Four design methods are illustrated with numerical examples:

(a) The common method, based on a linear stress distribution.

(b) Winkler's elastic bed method, based on a constant subgrade coefficient.

(c) The semi-infinite solid method, based on a correct stress distribution according to Boussinesq.

(d) Approximations.

The applicability of each method under the various soil conditions is discussed.

### Analysis of a simple braced barrel vault

Asoke Ghosh, *Indian Concrete Journal* (Bombay), V. 32, No. 8, Aug. 1958, pp. 278-281

Author gives an idea of the stress variation in two cases. Equations are offered which are said to simplify the computation and give answers reasonably close to the exact solution.

### Design of eccentrically loaded columns by the load factor method

J. D. Bennett, Concrete Publications, Ltd., London, 1958, 40 pp., $1.90

### Tables for the design of beams and slabs (by the load factor and elastic methods according to British Standard Code 114:1957)

Jacques S. Cohen, Concrete Publications, Ltd., London, 1958, 16 pp., $1

These concise yet comprehensive brochures have been prepared to simplify the design of reinforced concrete in accordance with the load factor method as recommended by British Standard Code No. 114 issued in 1957. Mr. Bennett's work gives 20 charts for the design of columns subjected to direct load and bending about one axis of symmetry, and reinforced symmetrically or unsymmetrically with mild steel and cold worked bars. Mr. Cohen's tables relate to the design of rectangular beams by the load factor method

and also by the elastic method, so that it is possible to see at a glance which is the most economical for any particular beam. Worked examples are given with both sets of tables.

### Numerical method for determination of critical buckling loads of two-hinged elastic arches

P. N. Chatterjee, *Journal of Technology*, Bengal Engineering College (Calcutta), V. 2, No. 2, Dec. 1957, pp. 145-149

Author's Summary

In the proposed method, the fundamental buckling configuration of a two-hinged arch is numerically determined by the procedure of successive approximation from which the critical horizontal thrust and the buckling load can be determined. The procedure, if used in conjunction with an energy method, yields accurate results even with relatively large finite difference intervals and few cycles of successive approximations.

A modification of the proposed procedure for determination of critical horizontal thrust for the second mode of buckling is also discussed. Some typical curves showing the variation of the values of critical horizontal thrust of two-hinged arches having constant moment of inertia but different rise-span ratios are presented.

### Pier slab system

*Concrete Construction*, V. 3, No. 8, Aug. 1958, pp. 6-9

Describes a residence foundation design called "intermediate pier floor slab construction." Important difference between this design and conventional slab-on-ground construction is that the concrete floor, reinforced with welded wire fabric, is a semi-structural slab somewhat similar to a bridge deck. The weight of the slab and its residential load is actually borne by the foundation walls and intermediate supporting piers which penetrate to undisturbed soil.

### Analysis of conical elements of revolution in flexure (Calcul des pièces coniques de révolution travaillant à la flexion)

Ma Min-Yuan, *Annales des Ponts et Chaussees* (Paris), V. 128, No. 1, Jan.-Feb. 1958, pp. 87-113; No. 2, Mar.-Apr. 1958, pp. 175-230

Reviewed by Aron L. Mirsky

Derivations, based on simplifying assumption that thickness is quite small compared to other dimensions, of expressions for quasi-circular plates and quasi-cylinders, and finally of formulas applicable to true cones. Author states his solution is rigorous yet simple to apply; comparison with the results of Taylor and Wenk indicates accuracy is also high. As examples of structural applications of cones of revolution, author cites floors and roofs of tanks, silos, kiosks.

### Simplified analysis of symmetrical frames and beams by moment distribution (Calcul simplifié des portiques et poutres symétriques d'après la méthode de Cross)

Serge Woinowsky-Krieger, *L'Ingénieur* (Montreal), V. 43, No. 172, Winter, 1957, pp. 28-30; V. 44, No. 173, Spring, 1958, pp. 34-36

Reviewed by Aron L. Mirsky

Just as a shortcut is possible if one end of a beam is hinged while the other end is fixed, so other simplifications are possible if structure and loading are symmetrical. Theoretical basis of simplifications is shown and application is illustrated by three worked examples.

## Pavements

### Design and construction of concrete roads overseas

Arthur R. Collins and Douglas R. Sharp, *Proceedings*, Institution of Civil Engineers (London), V. 9 (session 1957-58), Jan. 1958, pp. 23-64; discussion, Aug. 1958, pp. 553-568

An interesting and informative survey. Construction practices and design specifications in many countries, especially the United States, Germany, Holland, Belgium, and Denmark, are tabulated as well as discussed in the text, and there are 21 excellent photographs, particularly of construction equipment and practices. Authors reach the general conclusion that "while concrete road design and construction in various countries differ in some important details there are no differences in principle."

### Tests of concrete pavements on gravel subbases

L. D. Childs and J. W. Kapernick, *Proceedings*, ASCE, V. 84, HW 3, Oct. 1958, pp. 1800-1–1800-31

Authors' Summary

Concrete pavement slabs 8 in. x 12 ft x 18 ft joined by 1-in. round dowels were subjected to static loads to study the effect of various thicknesses of open-graded sand and

gravel subbases on the strength of the composite pavement structure. Deflections and strains in the concrete and pressures on the subbase and subgrade were measured for eight load positions when the slab was flat and when it was curled upward at corners and edges.

The experiments showed that open-graded sand and gravel subbases under flat slabs were more effective in reducing free-corner and free-edge deflections than in reducing strains. Under edge loads the subbases were less effective in reducing deflections and strains than under corner loads.

Pressures between the slab and subbase increased with subbase thickness but pressures on the subgrade diminished slightly as subbase thickness increased.

These tests indicate that the most practical formulas for the computation of stresses and deflections in concrete pavements due to loads are Westergaard's original 1926 equations for interior loads, his 1947 equations for edge loads, and Pickett's 1946 equation for stresses due to corner loads.

### The history of road tests

E. A. Finney, *Proceedings*, ASCE, V. 84, HW 3, Oct. 1958, pp. 1796-1–1796-21

Author's Summary

This report traces the construction of different types of road tests in the United States from the latter part of the 18th century to the recent AASHO Road Test in Illinois. Five distinct periods of road building history in this country are mentioned. Road testing is discussed in relation to these periods. Whenever possible, a brief description is given for each road test project including the date of construction, purpose, scope, and reported findings.

## Prestressed Concrete

### Determination of maximum amount of reinforcing steel in prestressed concrete beams (in Rumanian)

I. P. Mihai, *Revista Constructiilor si a Materialelor de Constructii*, (Bucharest) V. 10, No. 1, 1958, pp. 19-24

Reviewed by J. J. Polivka

Referring to previous studies by Freifeld, Ratz, Berdicevski, and Mihailov, on criteria regarding fissures and ruptures under various amounts of reinforcing in the compression and tension zones, theoretical derivations are compared with testing results. Further references are made to research and publications by Laupa, Siess, Newmark, Elstner, Hognestad, Janney, McHenry, and Chambaud.

### Report on floor or roof construction consisting of prestressed concrete double tee slabs

*Retardant Report* No. 4123-1, Underwriters' Laboratories, Inc.; *Journal*, Prestressed Concrete Institute, V. 3, No. 2, Sept. 1958, pp. 13-33

A fire endurance test was conducted on precast prestressed concrete double tee slabs. The assembly was tested without concrete topping on the slab. The precast prestressed units of the design tested, without normal concrete topping, showed stability under standard fire test conditions for 2 hr.

### Comparative analysis of specifications and practice on prestressed concrete used in various countries

Andre Paduart, *Proceedings*, Second Congress of Fédération Internationale de la Précontrainte, Amsterdam, 1955 (published 1958), pp. 757-775

An interesting comparison of specifications which exist in 7 countries. The only U. S. document analyzed was the criteria for prestressed concrete bridges issued by the Bureau of Public Roads. One is struck by the divergences which appear in the rules of the various countries on any given subject.

### Prestressed concrete cylindrical tanks

Leonard R. Creasy, *Proceedings*, Institution of Civil Engineers (London), V. 9 (session 1957-58), Jan. 1958, pp. 87-114; discussion, July 1958, pp. 419-431

Reviewed by Aron L. Mirsky

Discussion of design and construction practices, with especial attention to the effect of fixing the bottom of the wall at the base slab instead of using a joint detail permitting sliding. Costs and economic proportions for given capacities are briefly touched on.

### Prestressed pressure pipelines for Athens aqueduct

Peter J. Doanides, *Civil Engineering*, V. 28, No. 9, Sept. 1958, pp. 47-51

The 25-mile extension to the Athens aqueduct will utilize prestressed concrete pipelines in the pressure sections and in a 22,000-ft

siphon crossing a valley. The methods adopted for manufacturing, transporting, placing, and testing the 52-in. diameter prestressed pipe are described.

### Bond in pretensioned prestressed concrete

KEN-ITIRO ABE, *Proceedings*, Symposium on Prestressed Concrete and Composite Beams, Toyko, Sept. 4, 1954 (published Nov. 1955) pp. 22-26
AUTHOR'S SUMMARY

There is no general method to observe the bond in pretensioned prestressed concrete. The wire must be tensioned before the concrete is placed, and released after the concrete has developed sufficient strength. The draw out test can not be adopted in this case; because the wire will slip in the concrete when it is released if the length of embedment of the wire in the concrete is shorter than twice the bond length, and if longer, it will be unable to draw out the wire. A simple and practical method for observing bond is suggested, and some test results of this method reported.

### No winter shutdown for prestress plant in turning out bridge girders

BILL ALLEN, *Contractors and Engineers*, V. 55, No. 10, Oct. 1958, pp. 42-47

Describes operations of a prestressing plant with outdoor casting beds and how the plant adapted methods to meet severe weather conditions, including temperatures of 18 F. In spite of the extra work made necessary by cold weather, the plant was able to turn out 1120 ft of bridge girders per day.

### Proceedings of the Second Congress of the Fédération Internationale de la Précontrainte (in French, German, and English)

Fédération Internationale de la Précontrainte, London, 1958, 990 pp., $15 (available from Cement and Concrete Association, London)

Includes all papers, general reports, and discussions, presented at the Second Congress held in Amsterdam in 1955.

Covers the following topics: function of grouting and anchorages in the behavior of prestressed elements; experience and problems concerning the manufacture and use of steel for prestressing; progress of precast prestressed work in the factory and the assembly by prestressing on the site of precast units; moment distribution in statically indeterminate prestressed structures beyond the elastic phase; and influence of plasticity on the strength and instability of thin prestressed shells.

An additional section contains a comparative analysis of specifications in various countries and statements on the economic advantages of prestressed concrete.

(Many of the individual reports will be reviewed separately in the "Current Reviews" section as space allows.)

### Recommendations for end anchoring systems

STUVO (Netherlands) COMMITTEE ON END ANCHORING SYSTEMS, *Proceedings*, Second Congress of Fédération Internationale de la Précontrainte, Amsterdam, 1955 (published 1958), pp. 57-68
AUTHOR'S SUMMARY

Gives some considerations on the behavior of end anchoring systems (the Freyssinet and Magnel systems) in regard to slip. The theoretical considerations are supplemented by the results of a series of experiments.

The research resulted in the issuance of recommendations regarding the method of tensioning and anchoring, and the drawing up of a number of requirements to be stipulated for an anchoring system.

### Tests of optimum value of tension in prestressing cables (Recherche de la valeur optimum de la tension des armatures de précontrainte)

P. XERCAVINS, *Proceedings*, Second Congress of Fédération Internationale de la Précontrainte, Amsterdam, 1955 (published 1958), pp. 194-204

These tests show that the ultimate strength under fatigue measured on three series of beams with the same static ultimate strength increases with the permanent stress in the steel.

### Test of continuous prestressed beams in the plastic range and at rupture (Etude expérimentale de poutres continues précontraintes dans le domaine plastique et à la rupture)

GIORGIO MACCHI, *Proceedings*, Second Congress of Fédération Internationale de la Précontrainte, Amsterdam, 1955 (published 1958), pp. 501-543

Three prestressed continuous beams were tested to follow the distribution of moments in the various phases up to failure and to study their inelastic deformations. The ulti-

mate failure moment was determined both theoretically and experimentally; it was found that in no case did complete redistribution of moments occur.

### Ultimate strength of two-span continuous prestressed concrete beams as affected by tendon transformation and un-tensioned steel

P. B. Morice and H. E. Lewis, *Proceedings*, Second Congress of Fédération Internationale de la Précontrainte, Amsterdam, 1955 (published 1958), pp. 561-584

Results of laboratory tests on two-span prestressed beams indicate that the ultimate load does not vary under tendon profile transformation. They also show that the method of calculation for ultimate moments of resistance of sections outlined by the authors with un-tensioned steel provides results in fair agreement with those measured.

### Some creep tests on steels for prestressed concrete

G. M. Canta, *Proceedings*, Second Congress of Fédération Internationale de la Précontrainte, Amsterdam, 1955 (published 1958), pp. 178-193

The results of 40 creep tests are discussed in which the stresses varied over a wide range. The steels may be divided into three types: (1) cold-drawn steel wire; (2) cold-drawn and aged steel wires; and (3) hardened and tempered steel wire. It was generally found that under extremely high stresses normal cold-drawn wire behaves better than cold-drawn heat treated wire and quenched and tempered wire. The usefulness of relaxation data is emphasized.

An appendix adds the preliminary testing specifications for high tensile steel for prestressing issued in the Netherlands.

### Research on a prestressed concrete north-light shell structure

A. M. Haas, *Proceedings*, Second Congress of Fédération Internationale de la Précontrainte, Amsterdam, 1955 (published 1958), pp. 659-674

To investigate the influence of prestressing on the buckling of a north-light shell roof, tests were conducted on cardboard and reinforced mortar models.

Tests on cardboard models confirm the expectation that elastic buckling is not liable to occur. There are as yet no reliable formulas for inelastic behavior, and tests were therefore made on a one-tenth scale model. The model, of prestressed mortar, was loaded in stages up to failure. The factor of safety deduced for the actual structure was more than 3½.

## Properties of Concrete

### A comparison of cement strengths in mortars and concretes

Myron A. Swayze, *ASTM Bulletin*, No. 232, Sept. 1958, pp. 54-57

Data are presented on the relationship between: (1) tensile strengths of briquets and flexural strengths of 40-mm prisms; (2) between strengths of 2-in. cubes and modified 40-mm cubes; (3) between briquet strengths and concrete cylinder strengths; (4) between compressive strength of cylinders and flexural strength of prisms; (5) between compressive strength of concrete cylinders and mortar cubes; and (6) between concrete cylinders and modified mortar cubes.

The author believes that for the determination of the flexural and compressive strengths of cement mortars, the 40-mm prism should supplant both the briquet and the 2-in. plastic mortar cube.

### Effects of air-entraining agents on mortars (in Swedish)

Erik Hogberg, *Nordisk Betong* (Stockholm), V. 2, No. 3, 1958, pp. 313-320

Reviewed by Margaret Corbin

Optimum air content in mortar used in masonry and plaster work is defined as 10 to 15 percent by volume of fresh mortar. Air bubbles shall be stable and uniformly distributed; their mean diameter shall not exceed 0.2 mm. These conditions result in improved plasticity and workability of the mortar. The quantity of mixing water becomes reduced and the volume of fresh mortar somewhat increased, resulting in a slightly lower weight per unit volume of the mortar. Water retention is also improved and segregation less likely to occur. The strength of mortars with high cement content shows a slight decrease, especially when air humidity is low. The strength of mortar containing 2 parts lime, 1 part cement and 12 parts sand by volume remains practically unchanged; a higher lime content results in a

strength gain. Frost resistance is considerably improved. Air-entraining agents may have a harmful effect if added in excessively large quantities.

### Influence of shrinkage on the variation of the dynamic modulus of elasticity of concrete with rapid setting cement (in Rumanian)

FACAOARU and G. STANCULAESCU, *Revista Constructiilor si a Materialelor de Constructii* (Bucharest), V. 10, No. 1, 1959, pp. 15-19

Reviewed by J. J. POLIVKA

Referring to previous research and experiments by Dutron, Lucas, L'Hermite, Shank, Thomas, Whitney, Goodell, and theoretical investigations by Freyssinet, L'Hermite, Seikin, and Pickett, tests performed by authors at the Rumanian Research Institute are thoroughly described and discussed. Various relationships between the relative humidity of concrete and resonant frequency, and the effect of relative humidity (40 and 100 percent) on ultimate strength and modulus of elasticity during the period from 1 to 28 days are demonstrated in diagrams and tables. Of special interest are results concerning properties and preservation of concrete with rapid setting cements. Further references are made to V. Nicolau and sonic testing methods described by A. Hutter in *Bauplanung und Bautechnik*, Sept. 1955.

### Temperature damage in concrete and measures for its prevention (Temperaturschäden im Beton und Massnahmen zu ihrer Verhütung)

B. HAMPE, *Der Bauingenieur* (Berlin), V. 33, No. 1, Jan. 1958, pp. 6-10

Reviewed by ARON L. MIRSKY

A subject of more than passing interest, ably discussed with the aid of several object-lesson photographs. While remedies listed are not new, they bear reiteration.

### Stresses due to shrinkage in symmetrically reinforced ferroconcrete elements with reference to the nonlinear creep of concrete (in Russian)

M. M. MANUKYAN, *Izvestiya Akademii Nauk Armyanskoi SSR* (Erevan), V. 7, No. 3, 1954, pp. 19-32

APPLIED MECHANICS REVIEWS Aug. 1958 (W. Olszak)

An examination is made of the appearance and development of stress, appearing at times as the result of shrinkage of concrete in symmetrically reinforced ferroconcrete elements with reference to the nonlinear creep of concrete. External loading is not taken into account. The measure of shrinkage creep of concrete, as also the function characterizing the nonlinear relationship between the stresses and deformations of creep, is taken to be in the form advocated by Arutyunyan. The function $f(\sigma) = \sigma + \beta\sigma^2$ with the small parameter $\beta$ expresses a weak nonlinearity of this bond.

An analysis is performed of the solutions depending on the relation of the parameters characterizing the intensivity of the shrinkage and creep of the concrete. The calculations show that, depending on the percentage of reinforcement, the stresses which have arisen in the concrete and reinforcement increase rapidly in the first 2 or 3-month period and then tend to stabilize. In calculating nonlinear creep, these stresses may be approximately 50 percent larger than in the case of linear creep, and increase together with the increase in paramcter $\beta$.

### Corrosion of concrete by sulfuric acid

W. C. HANSEN, R. P. VELLINES and W. W. BRANDVOLD, *ASTM Bulletin*, No. 231, July 1958, pp. 85-88

Results of a study of methods designed to evaluate the resistance of portland-cement mortars to attack by sulfuric acid in sanitary sewers indicate that the attack may consist of corrosion by the acid and of expansion by reaction of calcium sulfate with tricalcium aluminate.

### Application of harsh concrete mixes and reground cement in fabrication of precast concrete (in Russian)

A. A. GORDEEV, *Beton i Zhelezobeton* (Moscow), June 1958, pp. 213-214

Reviewed by D. WATSTEIN

Recent studies in Hydraulic Structures Laboratory have shown that compressive strengths of 2700 to 3600 psi at 1 and 2 days can be obtained without steam curing by using harsh mixes with a water-cement ratio of 0.4 and a reground cement, or a blend of a regular portland cement and a reground cement. Reground cement was obtained by grinding a regular portland cement in a "vibro-mill" for 30 min. The specific surface of the reground cement was about 25 percent

greater than that of the original cement. The study was carried out for a range of curing temperatures from 59 to 77 F and a 6-bag mix.

## Leak-tight concrete for building tanks for storing crude oil and petrol (in Polish)

F. Lecznar and S. Oskroba, *Nafta* (Katowice), No. 7-8, 1957, pp. 197-204

Polish Technical Abstracts No. 2 (30), 1958

An investigation was conducted to find the relationships between the permeability and strength of concrete and: cement and aggregate, water-cement ratio, aggregate-cement ratio, grade of aggregate, and manner of mixing and curing concrete. The experiments showed that it is possible to prepare concrete impermeable to crude oil and petrol; however, if good results are to be obtained, the total surface of cement must be higher than the total surface of sand and aggregate grains.

## Survey of knowledge of plastic deformation and creep (Que savons-nous de la deformation plastique et du fluage du beton?)

M. R. L'Hermite, Supplement to *Annales de L'Institut Technique du Batiment et des Travaux Publics* (Paris), No. 117, Sept. 1957, Series Beton, Beton Armé (46), pp. 778-809

The author has analyzed a hundred French and foreign publications which have appeared during the past 40 years. He has established a synthesis of the present state of the problem of the plastic deformation and creep of concrete.

The effects of the composition of the concrete, of the method of conservation, of the volume under stress, the age of the concrete, the importance and duration of the loading and unloading are also considered. Creep under different stress conditions is also examined. The creep is an hydro-constrictive phenomenon dependent upon the movements of the water; it is covariant with the shrinkage. The state of deformation tends towards a value proportional to the stress with a minimum corresponding to the shrinkage. The deformation follows a law similar to that for viscosity:

$$\frac{d\,\Delta}{dt} = (\Delta\, m - \Delta)\, K(a, t)$$

The rate of creep appears to reduce proportionally as the volume required increases.

Testing and measuring apparatus are described and numerous experimental results are presented. A theory is proposed and presented in an impartial and objective manner, rather to satisfy the demands of those engaged in research and in experiment than to give a theoretical conclusion on the problem.

## Concrete on the west coast of Jutland. Part 2—Concrete durability

G. M. Idorn, *Progress Report* B 2, Committee on Alkali Reactions in Concrete, Danish National Institute of Building Research and Academy of Technical Sciences, Copenhagen, 1958, 12 D.kr. (25 percent reduction may be obtained by ordering the whole series)

This report is a continuation of *Progress Report* B 1 ("Current Reviews," ACI Journal, Oct. 1958, p. 519). It surveys three previous service records on the behavior of concrete structures on the west coast of Jutland prepared in 1914, 1924-27, and 1952. The information gained from these investigations is discussed. Types of deterioration included map cracking, warping, incrustations on concrete surfaces, abrasion and/or erosion, scaling, pop-outs, and complete disintegration. Emphasis is placed on attack by freezing and thawing, chemical effects of sea water, and alkali reactions.

## Cement-aggregate reaction in concrete of a Canadian bridge

E. G. Swenson, *Proceedings*, ASTM, V. 57, 1957, pp. 1043-1056

Author's Summary

A study was made of a concrete bridge deck which had expanded abnormally over a period of more than 25 years. The two deck sections involved, made with different aggregates, were found to have been affected by different types of cement-aggregate reaction. One section had been seriously affected by alkali-aggregate reaction involving mainly a phyllite in the coarse aggregate, but the actual expansion and extensive deterioration of the concrete was attributed in large part to freezing and thawing under adverse moisture conditions. A feature of this case was the failure of the mortar bar test to reveal potential reactivity in the source material in spite of other evidence. Some possible explanations are considered. In the

second section, abnormal expansion was probably due to the sand component which was shown to be reactive in the Conrow test, ASTM C 342.

### Experiences with strain measurements on prestressed concrete structures (Erfahrungen bei Dehnungsmessungen an Spannbetonbauwerken)

BERNHARD FRITZ, *VDI Zeitschrift* (Düsseldorf), V. 100, No. 2, Jan. 11, 1958, pp. 49-58
Reviewed by ARON L. MIRSKY

Author first discusses mounting and waterproofing of electric resistance gages and leads, and a study of gage creep over long periods of time due to the very large strains (4.5 percent) in prestressing wires stressed to 95 kg per sq mm. He then summarizes results of investigations of stresses and loss of prestress due to cable draping and friction, in both test girders and actual bridges. Correspondence between calculated and measured values is quite good.

## Structural Research

### Flexural strength of hollow unit concrete masonry walls in the horizontal span

A. R. LIVINGSTON, EDWARD MANGOTICH, and ROBERT DIKKERS, *Engineering Report* No. 29, 1958, Iowa Engineering Experiment Station, *Iowa State College Bulletin*, V. 57, No. 2, June 11, 1958, 18 pp., $.50

Results of a limited series of tests on the flexural strength and factors affecting the flexural strength of concrete masonry walls built with hollow units laid in running bond in the horizontal span. Variables included mortar composition, block moisture content, and wall thickness.

Variation in the mortar composition and strength had no significant effect on the values of modulus of rupture in the horizontal span or on the secant modulus of elasticity in flexure. Variation in the block moisture content at the time of construction did not noticeably affect the flexural strength or secant modulus of elasticity of 8-in. concrete masonry walls in the horizontal span.

The modulus of rupture of concrete masonry walls built of hollow units laid in running bond decreased appreciably with increasing wall thickness. The modulus of rupture of the wall panels tested appeared to be a direct function of the modulus of rupture of the individual units (4, 8, and 12 in. thick) of which the wall was constructed.

### Bend tests on Danish Tentor reinforcing bars, based on the German Code of Practice

AXEL EFSEN, Preprint of report presented at RILEM Symposium, Liége, Belgium, July 3-5, 1958, 8 pp.

A test series of 200 specimens covering a wide range of sizes appears to indicate that Danish Tentor reinforcing bars comply with the German code provisions for testing the bending capacity of transverse-ribbed reinforcing bars.

### Tests of shear connectors for composite beams

YOSHIO TACHIBANA, *Proceedings*, Symposium on Prestressed Concrete and Composite Beams, Toyko, Sept. 4, 1954 (published Nov. 1955) pp. 90-96

Failure tests of several shear connectors for composite beams were made, and 31 specimens were tested and compared in classifying 11 groups. Describes test procedure; shapes of the shear connectors are illustrated; materials used and results of tests are given in table form.

### Increase in concrete modulus of elasticity due to prestress and its effect on beam deflection

H. J. BRETTLE, *Constructional Review* (Sydney), V. 31, No. 8, Aug. 1958, pp. 32-35
AUTHOR'S SUMMARY

The creep strain of concrete is partially removed by preloading and the resulting concrete stiffened in the process. It follows that the concrete modulus of elasticity is increased and the subsequent deflection of a concrete structure thereby reduced. A relationship between prestress and both the ultimate strength and modulus of elasticity is experimentally established for axially loaded concrete cylinders. This relationship is used to predict the deflection of 26 pretensioned beams of dimensions 6 in. x 4 in. x 9 ft, and the comparison of the theoretical with the observed deflections shows reasonable agreement.

## General

### Destruction of a concrete rolling mill base by oil emulsion (Zerstörung eines Walzwerk-Betonfundamentes durch Öl-Emulsion)

HEINZ KAMINSKI, *VDI Zeitschrift* (Düsseldorf), V. 100, No. 20, July 11, 1958, pp. 851-854
Reviewed by ARON L. MIRSKY

Lubricating or cooling oil in conjunction with cooling or emulsifying water can cause disintegration of concrete machine bases. Attack is intensified by high temperatures involved in rolling mill operation. Paper discusses breakdown of oils during use and presents a case history of damage and method of repair. It is pointed out that the only real solution is prevention, which is best accomplished in the design stage.

### The contemporary curtain wall

WILLIAM DUDLEY HUNT, JR., F. W. Dodge Corp., New York, 1958, 462 pp., $12.75

A comprehensive review of the development and use of curtain walls with skeleton frame buildings. Author analyzes and evaluates these walls, their functions and malfunctions, component parts, materials, and installation. Pitfalls are pointed out, known solutions outlined, and accepted good practice specified. Tables and lists give data on insulating efficiency, fire resistance, and dimensional stability.

### Simplified construction of mixers and stirrers on the "Convergence-Divergence" principle

M. S. FRENKEL, *The Engineers Digest* (London), V. 18, No. 10, Oct. 1957, pp. 443-445
Reviewed by ARON L. MIRSKY

Previous articles by author (*eg., Engineers Digest*, Nov. and Dec., 1956, pp. 475-480 and 509-510; "Current Reviews," ACI JOURNAL, July 1957, p. 87) described the "Convergence-Divergence" principle and the basic construction of crushers and other equipment, based on this principle, which used two rotating components. The simplified application of this principle uses only one rotating component, with no apparent reduction in efficiency. A wide range of applications is described.

### Splicing of cold worked reinforcing bars by electric arc welding

AXEL EFSEN, Preprint of report presented at RILEM Symposium, Liége, Belgium, July 3-5, 1958, 12 pp.

This paper deals with electric arc welding of splices and anchorages for cold worked bars.

The joint is covered by an appropriate angle which serves as reinforcement and also as a practical means for executing the welding. The design of the joint is so arranged that little heating from the arc will take place in the zone where the bar is leaving the angle, and the heat will, therefore, inflict an insignificant damage on the cold worked material in the place where it could be damaging. The test showed that it was possible to make such joints in cold worked bars of the type used in the investigation.

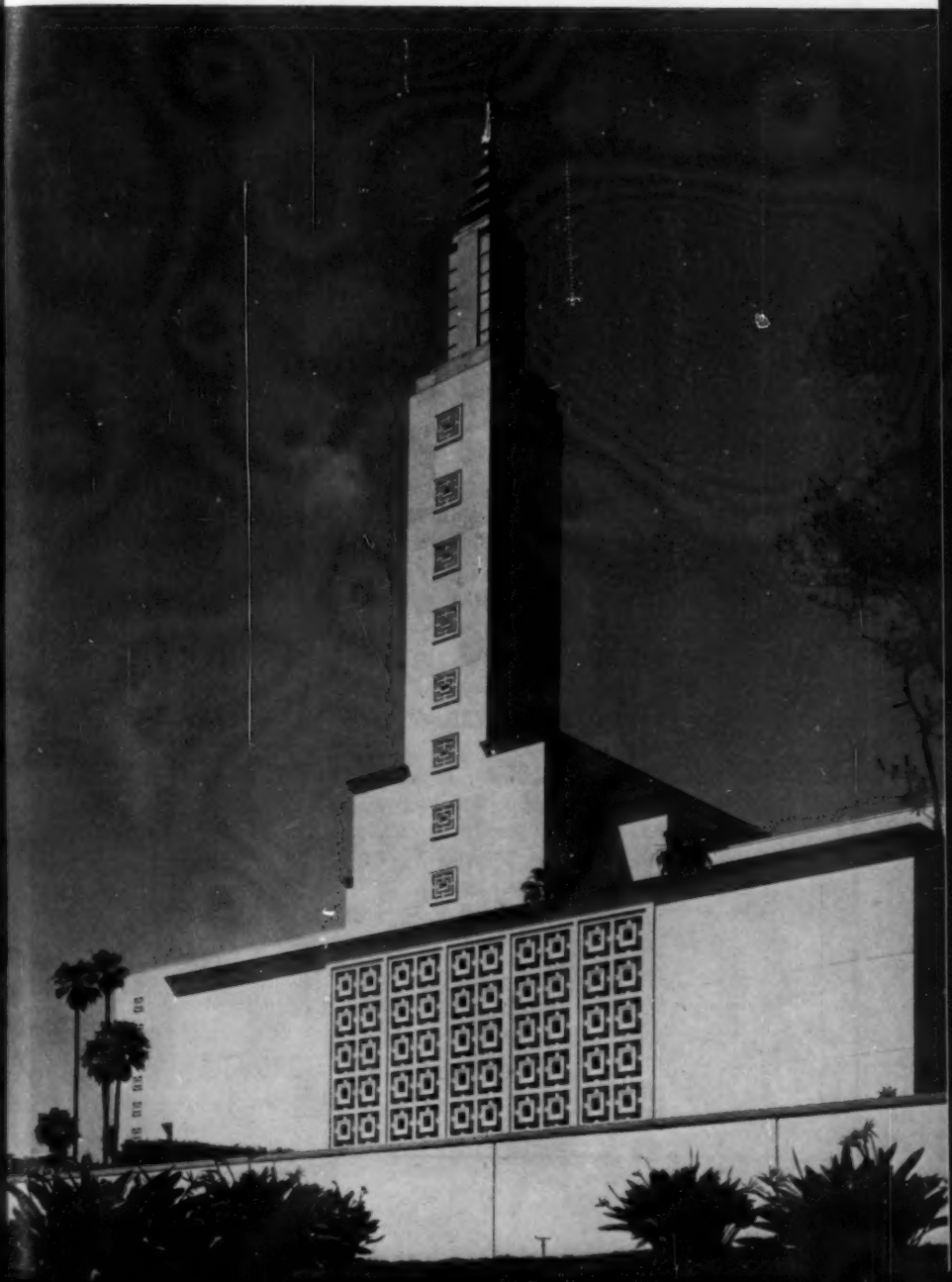
### Papers of the German Concrete Society meeting 1957

Deutscher Beton-Verein E. V., Berlin, Sept. 1957, 356 pp.
Reviewed by JOHN W. T. VAN ERP

Yearbook contains 12 papers by prominent experts. As usual the papers do not restrict themselves to concrete. The first paper for instance goes into the esthetic aspects of concrete construction and its influence on modern buildings, bridges, and prefabricated elements. Other papers deal with, reconstruction of Berlin, 10-year history of concrete construction in Germany, the 20-story concrete office building for the B.A.S.F. with much prestressing in it, papers on road construction, on dam structures for hydro-electric projects, on temperature damage to concrete and its prevention, building of an atomic reactor, and the development of a new concrete code. A new concept is presented in making use of statistical evaluation of concrete tests tied in with the bid prices of a large job. Also, construction features of the tallest concrete dam in the world, the Grande Dixense in Switzerland, and its large storage capacity, the second largest in the world.

# ACI News Letter

Vol. 30, No. 7 JOURNAL of the AMERICAN CONCRETE INSTITUTE January 1959



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News Letter Contents

January 1959

On the Cover—Glittering landmark on the Los Angeles skyline, the Church of the Latterday Saints rises 257 ft above its hilltop setting, making it visible for distances up to 25 miles. The entire building surface, more than 126,000 sq ft, is covered with precast concrete panels.

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# 1958 ACI Awards

## To honor Parsons, Mavis, Greaves, Brotchie, Rosenlund, and the late Joseph DiStasio, Sr.

1958 ACI award winners have been announced by the Board of Direction following approval of the Awards Committee report.

The Turner Medal goes to Douglas E. Parsons; the Kennedy Award to the late Joseph DiStasio, Sr.; the Wason Medal for Research to F. T. Mavis and M. J. Greaves; the Wason Medal for Most Meritorious Paper to John F. Brotchie; and the Construction Practice Award to Jack E. Rosenlund.

### Henry C. Turner Award to Douglas E. Parsons

*In recognition of notable achievement in the execution and direction of research on concrete, and in the furtherance of preparation and acceptance of concrete construction standards.*

This award, established in 1927, by the late Mr. Turner, ACI past president, is presented for notable achievement in, or service to, the concrete industry. The medal is of gold and is accompanied by an engraved certificate that recounts the medal's purpose.

Mr. Parsons, Chief, Building Technology Division, National Bureau of Standards, Washington, D. C., has contributed to the field of concrete through many lines. His technical contributions have been in the fields of masonry construction and reinforced concrete. He participated in the first International Building Research Congress (London, 1951) where his international reputation in the field was evidenced by an invitation to participate in pre-Congress discussion by selected members of the building research staffs of Great Britain and the Commonwealths.

He guided a comprehensive study at the National Bureau of Standards of means of controlling widths of cracks in reinforced concrete flexural members which produced a more reliable formula for estimating the width and spacing of tensile cracks,

thus permitting safeguards against corrosion in an economical manner.

Mr. Parsons recognized and stressed the importance of investigating the effect of length of shear span on the shear strength of reinforced concrete and, since the publication of the National Bureau of Standards results, more recognition has been given to this variable.

D. E. PARSONS

He has authored some 30 papers on reinforcement, compressive strength, shear strength, efflorescence, and water tightness of masonry walls; Mesnager hinges; cement water paints; and width and spacing of tensile cracks in reinforced concrete cylinders.

Active in ACI affairs since 1926, his record of achievement in Institute affairs presents an impressive picture. Culminating many years of activity with administrative and technical committees as well as the Board of Direction, Mr. Parsons was vice-president in 1943 and 1944, and president of the Institute in 1945.

In 1936 he was awarded the Wason Medal for Research for the paper "Tests of Mesnager Hinges" which he coauthored with A. H. Stang. In 1952 he received the Alfred E. Lindau award "for his many years of quiet perseverance in directing and coordinating development of standards of practice for reinforced concrete design." In 1952 Mr. Parsons also received the ASTM Award of Merit and in 1954 he was awarded the Department of Commerce Gold Medal for exceptional scientific attainments in the field of building research.

He is a graduate of Cornell College, Mount Vernon, Iowa, and active in numerous professional societies. In 1919, following 20 months in the armed service, Mr. Parsons joined the engineering department of the Iowa Railway & Light Co., where he was employed until joining the National Bureau of Standards in 1923. He was made chief of the Building Technology Division in 1947.

Exceedingly active in ACI technical committee work, Mr. Parsons is currently a member of the Executive Group of ACI Committee 115, Research; a member of ACI Committee 209, Volume Changes and Plastic Flow in Concrete; ACI Committee 318, Standard Building Code; ACI-ASCE Committee 323, Prestressed Reinforced Concrete; ACI-ASCE Committee 326, Shear and Diagonal Tension; and ACI Committee 335, Deflection of Concrete Building Structures.

## Henry L. Kennedy Award to Joseph DiStasio, Sr.

*In recognition of his outstanding contributions to the advancement of the objectives of the American Concrete Institute.*

This award, established in 1958, in memory of the late Henry L. Kennedy, past president of ACI and chairman of the Building Committee, is awarded for outstanding technical or administrative service to the Institute. The bases for selection of awardees of this award shall be outstanding activity or service that has enhanced the Institute's prestige, marked leadership in technical, administrative, or special committee work, or other distinguished service to the Institute. The award is in the form of a framed scroll.

The late Joseph DiStasio, partner in DiStasio and Van Buren, New York consultants, through development of design methods and devotion of time and effort in professional societies and code committees, effectively promoted and widened the field of reinforced concrete construction.

Mr. DiStasio, graduate of Columbia University, was a recognized authority on reinforced concrete. He wrote numerous technical papers dealing with the subject and received many awards for his outstanding contributions. Much of his work was done with and through the medium of ACI.

In 1955 he received the Concrete Reinforcing Steel Institute Award for Outstanding Contributions to Reinforced Concrete. The award was made in recognition of his firm's pioneering and development of the flat plate system. In 1957 the Brooklyn Institute of Design and Construction named Mr. DiStasio the "Engineer of the Year."

Mr. DiStasio joined the American Concrete Institute in 1920. He contributed to the JOURNAL pages and was an active participant in technical committee activities, particularly ACI Committee 318, Standard Building Code.

At the time of his death in September 1958, he was serving as chairman of Subcommittee 10 of Committee 318, and was a member of ACI-ASCE Committee 321, Design of Reinforced Concrete Slabs.

### Wason Medal for Research to F. T. Mavis and M. J. Greaves

This award goes to F. T. Mavis and M. J. Greaves for their paper "Destructive Impulse Loading of Reinforced Concrete Beams" published in the September, 1957, ACI JOURNAL.

Founded in 1917 by the late Leonard C. Wason, this award is bestowed to an ACI member or members for the most noteworthy research or discovery reported in an Institute paper within a given publication year.

F. T. Mavis is now dean of engineering, University of Maryland. He formerly was head of the department of civil engineering at Carnegie Institute of Technology. Prior to 1944 he served as department head at the State University of Iowa and Pennsylvania State University. He was educated at the University of Illinois and at the Technische Hochschule, Karlsruhe, Germany, where he was one of the first Freeman Traveling Fellows of ASCE. Dean Mavis has written extensively in hydraulic engineering, structural analysis, and applied mechanics.

M. J. Greaves is presently engaged as development engineer for Arthur G. McKee & Co., Cleveland. He formerly was professor of civil engineering at Utah State Agricultural College. As a major in the Panama Canal Department during World War II he supervised military construction projects. His experience has included many unusual problems in reinforced concrete design and stress analysis. Graduating from Cornell University in 1940, he received his PhD from Carnegie Institute of Technology in 1956.

### Wason Medal for Most Meritorious Paper to John F. Brotchie

John F. Brotchie receives this medal for his paper "General Method for Analysis of Flat Slabs and Plates" published in the July, 1957, ACI JOURNAL. This award was also established by Mr. Wason in 1917 and is presented for the most meritorious paper by a member or members of the Institute published in the latest volume of ACI *Proceedings*.

Mr. Brotchie graduated in civil engineering at Melbourne University, Australia, in 1950. Since then he has served with the Commonwealth Department of Works, Head Office, Melbourne. He spent his first year with the department on the design of roads and airfields, then turned his attention to the design of structures in steel, reinforced concrete, and prestressed concrete. He is now senior structural engineer in charge of the design of major structural projects.

### Construction Practice Award to Jack E. Rosenlund

Jack E. Rosenlund has been cited for his paper "Construction of the Dallas Memorial Auditorium" published in the October, 1957, JOURNAL. This construction award was founded in 1944 to help enrich the literature of that field of work and to honor the construction man—the man on the job—for his resourcefulness in translating design into a completed structure.

Mr. Rosenlund graduated from Southern Methodist University, Dallas, in 1944 and then served with the Civil Engineer Corps, U. S. Navy, during World War II. In 1948 he received his MS in civil engineering from the University of Minnesota.

Since 1949 Mr. Rosenlund has been a structural engineer with George L. Dahl Architects & Engineers of Dallas. In 1952 he became chief structural engineer for the organization.

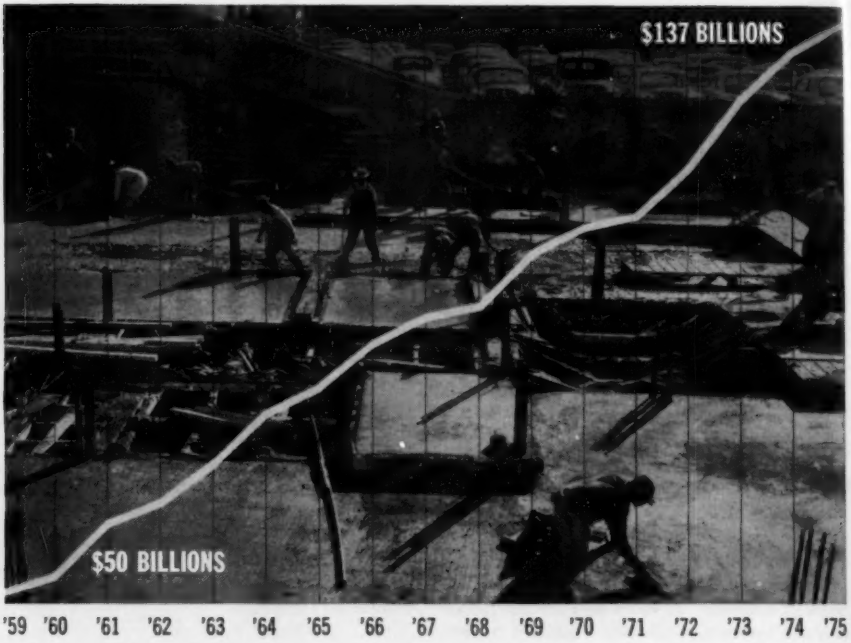
Presentation of the 1958 ACI awards will be made at the Awards Luncheon on Wednesday, February 25.

### *DIRECTORY INFORMATION*

**Have you changed employment or home address recently?**

**If so, please inform ACI Headquarters of these pertinant changes. Give both *old* and *new* information.**

**This information will shortly be included in the 1959 Institute Directory and any delay may result in an incorrect entry.**

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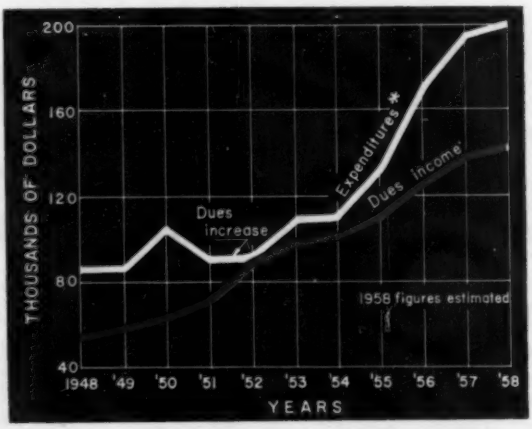
# RISING COSTS OUTSTRIP ACI GROWTH

Last month's editorial presented the general background reasons which led the Board of Direction, after a study of the ACI financial status, to accept a petition to raise dues. The petition will now go to the convention floor in February, 1959, and, if favorably acted on, to a vote of the Institute membership shortly thereafter.

For the 3 years immediately following the last dues increase in 1951 there was a sizable margin between income and expenses, allowing the rebuilding of Institute financial reserves which had been depleted because of delaying too long a needed dues increase. The additional funds also allowed the further development of the employee retirement fund and an increase in services, notably the addition of a technical director and needed general staff personnel. In the past 4 years, inflation of cost of materials and services has been the principal factor in causing an increasing gap between expenses and income from dues as indicated on the accompanying chart. As a matter of fact, had staff vacancies been filled immediately, a deficit would have resulted

**AN INCREASING GAP between dues income and expenditures* is evident during the period following the last dues increase in 1951. Inflation of cost of materials and services in the past 4 years has been the principal factor**




*Not including expenses chargeable to Publication Sales and Advertising departments

from 1958 operations. Direct cost of member services which include the JOURNAL, Directory, awards, and certificates was about $23,500 in 1951 against about $81,000 for 1958, a 245 percent increase. Personnel costs (wages, retirement, social security, and health insurance) were $54,000 in 1951 compared to $100,000 in 1958, up 85 percent. Other elements of Institute operating expenses went up in similar fashion.

It might be asked—and logically—where the money has come from to cover these increases in costs and why more is not forthcoming to carry on in the same way. As shown last month, expenditures for ACI operation have gone up from $19.80 per member in 1951 to $25.70 in 1958. The dif-

ference between dues paid and expenditures has been made up from revenues from publication sales and advertising which have increased 123 percent and 83 percent, gross, during the period. However, these sources of support to member service are now at a level which is not expected to rise significantly in the near future.

Does the new ACI headquarters building have any significant effect on the ACI financial situation necessitating a dues increase? Trendwise, the data charted are not significantly affected by the change from cramped inadequate rented quarters to the more efficient new ACI headquarters. This is principally because the new building was in large measure financed by contributions, so that current budgets have only to cover payments on a small mortgage plus operating expenses and maintenance. Since there is more than twice as much space in the new office the dollar cost is greater but still amounts to only 3.5 percent of the total 1958 budget (4.8 percent in 1951).

Rising cost of purchased materials and services have been largely responsible for the disappearing margin between ACI total income and expenses. Further inflation of costs appears almost inevitable. Therefore, to maintain present services to members, not to mention new ones which will be needed, will require more income. Since other sources of revenue—subscriptions, publications, and advertising—have had recent rate increases, ACI can look now only to its members to provide the funds to keep ACI the leading organization in this field which is so vital to our national welfare. Your favorable vote on the forthcoming dues proposition will make it possible.

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# ACI technical committee appointments

Listed below are committee members who have recently accepted appointments to ACI technical committees. Included are *new* appointments only.

### Committee 115, Research

A. R. Collins
Cement and Concrete Association
London, England

F. M. Lea
Scientific and Industrial Research
Building Research Station
Garston, Herts, England

Inge Lyse
Laboratories du Batiment et des Travaux Publics
Paris, France

Niels Munk Plum
Danish National Institute of Building Research
Copenhagen, Denmark

Mirko Robin Rõs
Consulting Engineer
Zürich, Switzerland

Hubert Rüsch
Institute of Technology
Munich, Germany

Eduardo Torroja
Instito Technico de la Construccion y del Cemento
Madrid, Spain

### Committee 318, Standard Building Code

Paul F. Rice
Concrete Reinforcing Steel Institute
Chicago, Ill.

*Continued on p. 22*

CALIFORNIA here we come!

# 55th Annual Convention Los Angeles, Feb. 23-26

An outstanding technical program, special entertainment highlights, and an opportunity to enjoy a midwinter vacation in sunny California are three attractions of the 55th annual ACI convention scheduled to be held at the Statler Hilton Hotel, Los Angeles, February 23-26.

The convention opens Monday, February 23, with most of the first two days devoted to technical committee meetings. First general session is scheduled for Tuesday afternoon, February 24, and will consider revision of ACI Bylaws, and new and revised standards. Six sessions will be held beginning Wednesday and running through Thursday noon, to be followed by the annual research session on Thursday afternoon.

## Technical Sessions

The first general session Tuesday afternoon will include possible revisions of ACI Bylaws, and new or revised ACI Standards. The standards to be presented for adoption cover hot weather concreting; proportioning of lightweight aggregate structural concrete; and measuring, mixing, and placing concrete. Concurrent sessions Wednesday morning will feature construction and special problems in concrete.

Concurrent sessions Wednesday afternoon will be on design and analysis, and concreting materials and methods. Thursday morning sessions will feature design research, and products and precast elements.

The technical program will culminate Thursday afternoon with the annual research session, an open meeting of ACI Committee 115. A group of short papers will be presented on research techniques and preliminary results of investigations now in progress.

The annual awards luncheon, a feature of the ACI meeting, will be held on Wednesday, February 25.

## Exhibit

An exhibit will be held in conjunction with the convention and, as an added feature, the exhibits committee and the Southern California Chapter of ACI are sponsoring a student competition for concrete designs and applications for students in southern California universities. Four cash awards totaling $400 will be made at the awards luncheon. Entries will be on display at the convention.

## Entertainment Program

Augmenting the fact-filled technical sessions, an impressive schedule of entertainment events has been arranged by the local entertainment chairman, C. Taylor Test. Ladies have been given prime consideration in the entertainment planning, with two activities planned solely for them, in addition to other events for all convention participants.

Both delegates and wives are invited to make the trip to world-famous Disneyland on Friday, February 27, following the conclusion of the official convention program. All conventioneers are invited to the social hour slated for Wednesday night, February 25, which will have a "Pacific island" setting.

In addition to these planned events, tickets to popular television shows and reservations for famous Los Angeles and Hollywood restaurants and night spots will also be available for convention attendants. There will be ample nonscheduled time to take in the sights of southern California, truly an area of "things to see and do" for visitors.

## Local Planning Committee

The Los Angeles local committee assisting chairman Sam Hobbs, Secretary-Treasurer,

ACI Southern California Chapter, includes: John McNerney, Portland Cement Association; Byron P. Weintz, Consolidated Rock Products; William F. Norton, Ceco Steel Products Corp.; Taylor Test, Riverside Cement Co.; Glenn C. Thomas, Thomas Concrete Accessory Co.; Lewis K. Osborn, Kistner, Wright, and Wright, architectural firm; and Ross Adams, Portland Cement Association.

The ladies contingent is headed by Mrs. Taylor Test as chairman with Mrs. Ray Cooley assisting her. Hostesses for the buncheon include: Mrs. Charles Pankow, head hostess, Mrs. Ernest Maag, Mrs. Byron Weintz, Mrs. Sam Hobbs, Mrs. Ross Adams, Mrs. Harold Olmstead, Mrs. Joe Sheffet, and Mrs. John Driskell. Hostesses for the fashion luncheon will be: Mrs. Marvin Kudroff, head hostess, assisted by Mesdames Henry Layne, R. W. Blair, W. C. Bryant, Robert E. Tobin, Felix Kulka, and John McNerney.

# CONVENTION PROGRAM

## TUESDAY AFTERNOON, FEBRUARY 24

### First General Session

**Proposed Revision of Institute Bylaws**

**Proposed Standards or Revision of Standards**
**Committee 605, Hot Weather Concreting**
**Committee 613, Proportioning of Lightweight Aggregate Structural Concrete**
**Committee 614, Measuring, Mixing, and Placing Concrete**

**Committee Report:**
**Committee 326, Shear and Diagonal Tension**

## WEDNESDAY MORNING, FEBRUARY 25

### Construction Session

**Erection of Precast Concrete**—J. L. Peterson, J. L. Peterson, Inc., Los Angeles

**Joinery of Precast Elements**—W. Howard Gerfen, consulting structural engineer, San Marino, Calif., and John R. Anderson, consulting structural engineer, Pasadena, Calif.

**Concrete Pavements on Cement-Treated Subgrades in California**—J. W. Trask, California Division of Highways, Marysville

**Qualification Plan for Ready-Mixed Concrete Plants**—C. E. Proudley, North Carolina State Highway Commission, Raleigh

**Cast-in-Place Concrete Pipe**—Louis Kristoff, Corps of Engineers, Sacramento, Calif.

### Special Problems in Concrete

**Properties of Shielding Concrete**—James Henrie, Atomics International, Canoga Park, Calif.

## *LADIES*

**Special features for the ladies include a get-acquainted bruncheon on Tuesday morning, second day of the convention. Ultra-deluxe attraction is a fashion luncheon at the Beverly Hilton Hotel, Beverly Hills, with the fashion show staged by Walter Plunkett, Academy Award-winning designer for Metro-Goldwyn-Mayer Studios. This event is scheduled for Thursday noon, February 26. Mr. Plunkett will display current fashion designs, along with his award-winning creations. The luncheon will be followed by a short tour of the Beverly Hills area.**

**Design and Performance of Special Purpose Blast Resistant Structures**—R. A. Williamson, Holmes and Narver, Inc., Los Angeles

**Influence of Grade of Steel on Blast Resistance of Reinforced Concrete Beams**—W. A. Shaw and J. R. Allgood, U. S. Naval Civil Engineering Laboratory, Port Hueneme, Calif.

**Surface Cooling of Mass Concrete to Prevent Cracking**—Roy W. Carlson, consulting engineer, Berkeley; Don Thayer, California Department of Water Resources, Sacramento

## WEDNESDAY AFTERNOON, FEBRUARY 25

### Design and Analysis Session

**Design Prestressed Concrete Lift Slabs for Deflection Control**—Edward K. Rice and Felix Kulka, T. Y. Lin and Associates, Van Nuys, Calif.

**Effect of Longitudinal Forces on a Portal Frame Supporting a Highway Bridge Deck**—Tung Au, Carnegie Institute of Technology, Pittsburgh, and Thomas D. Y. Fok, Youngstown University, Youngstown, Ohio

**Prestressed Concrete Roofs for Grandstands in Caracas, Venezuela**—Henry M. Layne, structural engineer, Los Angeles, and T. Y. Lin, T. Y. Lin and Associates, Van Nuys, Calif.

**Laboratory Research on Pavements Continuously Reinforced with Deformed Bars**—J. L. Waling and M. J. Gutzwiller, Purdue University, Lafayette, Ind.

**Stresses and Deflections in Pavements Continuously Reinforced with Deformed Bars**—J. L. Waling and M. J. Gutzwiller, Purdue University, Lafayette, Ind.

### Concreting Materials and Methods

**Evaluation of Concrete and Mortar Mixes**—W. A. Cordon, Utah State University, Logan

**Shotcrete: Properties and Application to New Buildings**—Stanley Zynda, Gunite Contractors Association, Los Angeles

**Controlling Tremie Concrete with Admixtures**—J. Wayman Williams, Sika Chemical Corp., Passaic, N. J.

**Concrete and Concrete Materials for Glen Canyon Dam**—W. H. Price, L. P. Witte, and L. C. Porter, U. S. Bureau of Reclamation, Denver

## THURSDAY MORNING, FEBRUARY 26

### Design Research

**Effect of Bar Cutoff on Bond and Shear Strength of Reinforced Concrete Beams**—Phil M. Ferguson, University of Texas, Austin, and Farid N. Matloob, Mosul, Iraq

**Shear Strength of Two-Span Continuous Reinforced Concrete Beams**—Jose J. Rodriguez, Albert C. Bianchini, University of Illinois, Urbana; Ivan M. Viest, AASHO Road Test, Ottawa, Ill.; and Clyde E. Kesler, University of Illinois

**Torsional Resistance of Reinforced Concrete**—George C. Ernst, University of Nebraska, Lincoln

**Behavior of a Continuous Concrete Slab Prestressed in Two Directions**—A. C. Scordelis, T. Y. Lin, and R. Itaya, University of California, Berkeley

### Products and Precast Elements

**Precast and Prestressed Folded Plate Slabs**—H. H. Edwards, Leap Concrete, Inc., Lakeland, Fla.

**Plastic Forms for Architectural Concrete**—J. A. Hanson, Portland Cement Association, Chicago

**Exposed Aggregate Type of Decorative Panels**—C. D. Wailes, Jr., Wailes Precast Concrete Corp., Sun Valley, Calif.

**Glazed Face Concrete Block**—S. H. Westby, Portland Cement Association, Chicago

**Linings for Concrete Pipe and Structures under Sewage Conditions**—C. G. Munger, Amercoat Corp., South Gate, Calif.

## THURSDAY AFTERNOON, FEBRUARY 26

### Annual Research Session

This convention feature differs from general convention sessions in that the information is not released for publication. A group of short papers will be presented on research techniques and preliminary results on a number of current projects not yet sufficiently advanced to justify final reports.

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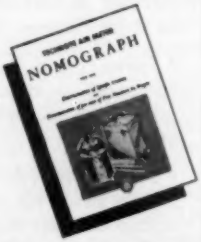


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This book is an up-to-date introduction to the fundamentals of reinforced concrete, including pertinent information on the physical behavior of reinforced concrete members. The emphasis throughout the book is on the ultimate strength concept and the manner in which beams and members fail. This new design technique utilizes proven strengths established from earlier research. In 1956 the ACI Building Code incorporated provisions for ultimate strength as an alternative method. Already rapidly gaining favor in the U. S. and abroad, the ultimate strength theory will undoubtedly become the predominant design procedure.

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**ACI-19**

# Positions and Projects

## Oklahoma ACI chapter moves to elect officers

Robert P. Witt, Oklahoma State University, Stillwater, presided as chairman at the Oklahoma ACI chapter meeting held November 24.

Professor Witt made an official announcement that the organization of the Oklahoma Chapter and the proposed bylaws for the chapter had been conditionally approved by the national organization's Board of Direction.

Harold Wenzel of the Dolese Co., Oklahoma City, was appointed chairman of the nominating committee to select a slate of nominees for the newly organized chapter to be elected by letter ballot. Members appointed to the nominating committee were: James Sprouse, The Master Builders Co. and J. C. Brown, Harter Marblecrete Stone Co., Inc., both of Oklahoma City; and Richard Pool of the University of Oklahoma, Norman.

## Northern California ACI chapter plans first annual meeting

The Institute board of direction has approved the formation of the Northern California Chapter of ACI.

The chapter met in San Francisco on November 12 with 75 in attendance and S. D. Burks of Dewey and Almy Chemical Co., San Leandro, serving as chairman. Officers were elected to serve until the first annual meeting which the sponsoring committee is hoping to hold in February.

T. Y. Lin, University of California, Berkeley, served as principal speaker for the meeting. He spoke on "Observations on Concrete in Russia."

Now that the chapter is official and the bylaws have been accepted the next step on the agenda is a membership drive. Harry Thomas, Pacific Gas and Electric Laboratory, Emeryville, is chairman of the membership committee with Edward L. Howard, Jr., Pacific Cement and Aggregates, Inc.; Max Neumann, Woodward, Clyde and Associates, Oakland; and Lewis H. Tuthill, California State Department of Water Resources, Sacramento, as committee members.

## Appointments made to ACI Technical Activities Committee

Walter J. McCoy of the Lehigh Portland Cement Co., Coplay, Pa., has been reappointed chairman of the Technical Activities Committee for a 1-year term. Clyde E. Kesler, University of Illinois, Urbana, and Bryant Mather, Waterways Experiment Station, Jackson, Miss., have been reappointed to membership on the committee for 2-year terms effective February 25, 1959. J. J. Shideler, Portland Cement Association, Skokie, Ill., was newly appointed to the committee for a 2-year term effective the same date.

## Philleo named chairman of ACI Committee 214

Robert E. Philleo, Office, Chief of Engineers, Washington, D. C., has been appointed as chairman of ACI Committee 214, Evaluation of Results of Strength Tests of Concrete, for a 3-year term.

Mr. Philleo joined ACI in 1947 and at the present time is secretary of ACI Committee 207, Properties of Mass Concrete, and secretary of ACI Committee 621, Aggregates.

## J. K. Gannett retires

J. K. Gannett, retired January 1st after 43 years with the Austin Co. He joined Austin in Cleveland in 1915 as assistant chief engineer and for the past 15 years has served as vice-president in charge of all engineering and research for the firm.

A 1910 civil engineering graduate of the University of Michigan, he is credited with many important "firsts." In 1953 he received a citation as a Distinguished Alumnus "in recognition of his outstanding achievements and contributions to the development of the field of engineering."

A native of Wellington, Ohio, Mr. Gannett is active in a number of technical societies and is a 20-year member of ACI.

# Precast Concrete Components Lifted and Placed with Richmond Lifting Inserts

Precast wall slab being tilted into place by means of Richmond Lifting Inserts and Lifting Brackets.

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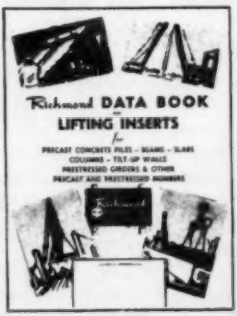


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## PCI elects officers and directors for 1958-59

At the annual Prestressed Concrete Institute membership meeting, September 22, 11 new officers and directors were elected for 1958-59.

Peter J. Verna, Jr., Concrete Materials, Inc., Charlotte, N. C., assumes the presidency; Philip E. Balcomb, American-Marietta Co., Chicago district, was elected vice-president and Jacob O. Whitlock of the Midwest Prestressed Concrete Co., Springfield, Ill., was elected secretary-treasurer.

Directors include holdovers, Douglas P. Cone, Florida Prestressed Concrete Co., Inc., Tampa, Fla.; T. Y. Lin, University of California, Berkeley; J. Ashton Gray, Dura-Stress, Inc., Leesburg, Fla.; George W. Ford, R. H. Wright and Son, Inc., Fort Lauderdale, Fla.; and Randall M. Dubois, Freyssinet Co., New York.

Newly elected directors include: Ben C. Gerwick, immediate past president, Ben C. Gerwick Co., Inc., San Francisco; R. O. Kasten, Union Wire Rope Corp., St. Louis; and John K. Zimmerman, Pennsylvania Prestress, Inc., York, Pa.

## Shelberg named to highway construction coordination group

Edward Shelberg is one of five state highway department engineers named as expediters to coordinate construction of major expressway routes in Michigan's $1¼ million, 5-year highway construction program. Mr. Shelberg will be assigned to the Detroit-Muskegon (U. S.-16) expressway.

## A. Warren Simonds retires from Bureau of Reclamation

A. Warren Simonds, head of the foundations and structural behavior section in the Bureau of Reclamation engineering offices at Denver retired November 30, completing more than 31 years of service with the Bureau.

As an authority on grouting techniques for dams, Mr. Simonds has participated in the design of numerous structures for water resources development throughout the 17 western states. During the early 1930's he pioneered in and directed the testing of structural models of dams including Hoover, Grand Coulee, Norris, and Gibson Dams. Techniques he developed in structural model construction are being applied to model tests of Glen Canyon Dam now being constructed by the Bureau in northern Arizona.

He has gained international recognition, having served as an engineering advisor to the governments of Japan, Formosa, New Zealand, Australia, Ethiopia, Costa Rica, and Lebanon.

Mr. Simonds is the author of numerous technical papers on foundation behavior and grouting. He is a member of the American Society of Civil Engineers and was the winner of the Society's Thomas Fitch Rowland Prize in 1954. He is past president of the Professional Engineers of Colorado and a member of the American Concrete Institute as well as a number of other technical societies.

## Lehigh announces executive appointments

Lehigh Portland Cement Co., Allentown, Pa., announce the election of Philip E. Carlson as assistant vice-president in charge of manufacturing. Paul E. Anderson has been elected treasurer of the company.

*Before you pour*

get longer life
for concrete with

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More and more contractors are realizing that it pays to install Welded Wire Fabric wherever concrete construction is called for. They have found through experience that fabric reinforcement is superior in several important ways:

- **Minimizes warping and heaving caused by varying temperatures and moisture content**
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- **Lengthens life of concrete, preserves its smooth, attractive surface, and keeps maintenance costs to an absolute minimum**

You get all these advantages when you use Clinton Welded Wire Fabric, plus the knowledge that it's a quality product made and backed by a nationwide steel producer. Readily available in both the East and West, in all popular sizes and lengths, Clinton Welded Wire Fabric is your easiest, most dependable way to assure longer life for concrete.

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5814

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## Annual Raymond Award goes to Martin S. Kapp

Martin S. Kapp has won the first annual Alfred A. Raymond Award of $1000 for his paper describing a new technique of soil consolidation, according to a recent announcement by Maxwell M. Upson, chairman of the board of Raymond International, Inc., sponsor of the award. Named after the firm's founder, the award was established "to encourage originality in research and development in the field of foundation engineering."

Mr. Kapp, soils and foundation engineer for the Port of New York Authority, described how a large area of unstable soil was consolidated by lowering the water table temporarily with wellpoints. The method was used instead of a conventional surcharge fill.

## Szilard faculty member at Colorado State University

Rudolph Szilard is now affiliated with Colorado State University, Fort Collins, in the capacity of associate professor. He is teaching structural engineering courses and doing research work in the same field.

Professor Szilard is a member of ACI Committee 338, Torsion.

## Huron Portland Cement to complete new plant in 1959

Approval for the completion of a new cement distributing plant at St. Joseph, Mich., was granted recently by the board of directors of the Huron Portland Cement Co.

The new facility, located on the St. Joseph River, will be the company's thirteenth distributing plant, according to Paul H. Townsend, president.

The new plant, located on a five acre site, will have ten silos with a storage capacity of 150,000 barrels. Bulk and package loading facilities for trucks and railroad cars will be located directly beneath the silos. Like the other Huron plants located throughout the Great Lakes region, the new St. Joseph plant will be supplied with cement from the company's mill at Alpena, Mich., by Huron's own fleet of six self-unloading ships.

Completion of the plant is scheduled for early fall, 1959.

## Dobbins elected PCA board chairman

Cris Dobbins, president of the Ideal Cement Co., Denver, was elected chairman of the Board of Directors of the Portland Cement Association at its annual meeting in Chicago in November.

Mr. Dobbins, who has served on the board of directors and as a member of various committees since 1948, succeeds George E. Warren, president of Southwestern Portland Cement Co., Los Angeles.

At the same time Hugh D. Barnes was appointed assistant vice-president and Anthony G. Sabato as auditor.

Mr. Barnes, who recently was appointed supervisor of field promotion after having served for 2 years as manager of the PCA western regional office in Los Angeles, will direct and have complete authority and responsibility for all operations of the Association's 32 district and 6 regional offices.

Mr. Sabato, who has served as assistant auditor since 1953, will supervise the association's accounting and auditing operations.

## Mulliken joins Sika

Robert B. Mulliken has joined the Pittsburgh district office of the Sika Chemical Corp. to furnish technical information and service to architects, engineers, and contractors on Sika additives and Sika materials for waterproofing. As sales engineer, he will service western Pennsylvania, West Virginia, and eastern Ohio.

## 1959 American Society for Engineering Education awards

The American Society for Engineering Education has opened nominations for the four principal awards in engineering education to be presented by the society during its 1959 annual meeting in Pittsburgh, June 15-19, 1959.

The Lamme Award, a gold medal given annually since 1928 to honor "distinguished achievements which contribute to the advancement of engineering education;" the George Westinghouse $1000 Award "to recognize and encourage outstanding contributions to the teaching of students of engineering;" the Vincent Bendix Award for "ad-

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**Designers and builders of fine structures such as this $10-million lightweight concrete skyscraper employ POZZOLITH to obtain unmatched performance and lowest cost-in-place concrete**

POZZOLITH is widely used in lightweight aggregate concrete. In this case—the mix weighed not more than 102 pounds per cubic foot. Compressive strength averaged 3750 psi with 6 sacks of cement per cubic yard.

POZZOLITH improves concrete quality and reduces the cost of concrete in place by providing three basic controls: control of water content—makes possible lowest water content for a given workability, control of entrained air, and control of rate of hardening.

Any one of our more than 100 skilled fieldmen will be glad to discuss and demonstrate the benefits of these POZZOLITH controls for *your* projects.

Illuminating Building, Cleveland, Ohio. In-construction and completed views.

Architects: Carson & Lundin, New York City.

Engineers: McGeorge-Hargett & Assoc., Cleveland.

Contractor: George A. Fuller Co., New York City.

Pozzolith Ready-Mixed Concrete—Cleveland Builders Supply Co.

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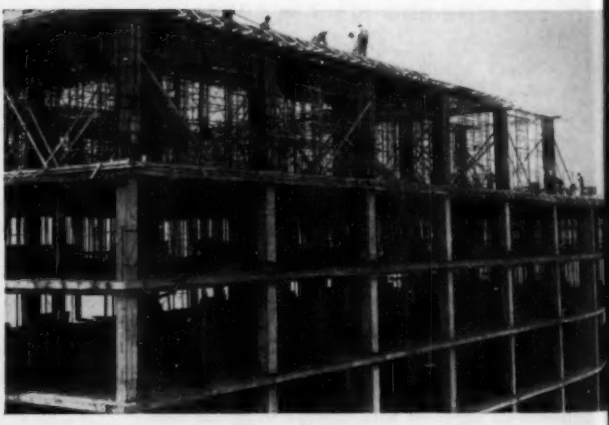

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vancement of engineering research program in colleges and universities;" and the $1000 Curtis W. McGraw Research Award to recognize and encourage "early achievements by engineering college research workers under 40 years of age."

In addition, the Society will award $300 in prizes for the best papers written by ASEE members under 36 years of age and appropriately entered in the national contest. ASEE's Technical Institute Division also will present the James H. McGraw Award of $500 to a leader in technical institute education.

## Ireland elected head of National Slag Association

Charles W. Ireland was elected president of the National Slag Association at the 41st annual meeting of the society.

Mr. Ireland is president of the Vulcan Materials Co., Birmingham, and brings to his new post many years of experience in slag processing and marketing.

## NSPI conducts second annual convention

The second annual convention of the National Swimming Pool Institute was held November 29 to December 2 at the Los Angeles' Hotel Ambassador with some 2000 delegates in attendance.

The NSPI is a nonprofit association organized 3 years ago to represent builders, suppliers, service representatives, architects, engineers, designers, and pool owners and operators. Objectives of the organization are to raise industry standards and ethics, establish uniform standards for design, construction and equipment, and to achieve uniformity in public regulations applying to swimming pool construction and operation.

## Morrison elected president of Peerless Cement

Robert J. Morrison, vice-president and secretary of American Cement Corporation, has been elected president of Peerless Cement Co., a division of American, with headquarters in Detroit.

Mr. Morrison joined Peerless as a plant engineer in 1945. He became assistant to the president in 1953 and executive vice-president in charge of administration in 1956. He will continue as a vice-president and a member of the board of directors and the executive committee of American.

## Commerce Department announces technical translation directory

The Department of Commerce began the publication in January of a periodical planned to serve as a central source of information in the United States on Russian and other technical translations available to science and industry.

The periodical, *Technical Translations*, will be published twice a month by the Office of Technical Services, in cooperation with the Special Libraries Association. It will list and abstract translated material available from U. S. Government sources, SLA, cooperating foreign governments, educational institutions, and private sources.

The Foreign Technical Information Center was established last year in the U. S. Department of Commerce to provide American science and industry with access to translations of a large amount of Soviet technical information. The center is a part of the Office of Technical Services, Business and Defense Services Administration, under John C. Green, director of OTS.

The services of the Foreign Technical Information Center include publication of abstracts of all articles appearing in 141 Soviet technical journals, translations of important sections of *Referativnyy Zhurnal* (the Russians' own abstract journal), and a semimonthly review of various areas of Soviet science compiled by the Central Intelligence Agency. Abstracts of each issue of the 141 journals may be purchased from OTS on a subscription or single issue basis, as may CIA's *Scientific Information Report*.

JOURNAL readers may be interested in articles abstracted in English from the following Russian civil engineering publications. Abstract cards may be purchased on a subscription basis.

Monthly publications: *Concrete and Reinforced Concrete; Hydraulic Engineering Construction; Hydraulic Engineering and Soil Improvement* and *Mechanization of Construction*. Bimonthly publication: *Cement*.

## ACI Appointments

*Continued from p. 8*

### Committee 325, Structural Design of Concrete Pavements for Highways and Airports

Harry D. Cashell
Bureau of Public Roads
Washington, D. C.

Alvin R. Schwab
American Steel and Wire Division
U. S. Steel Corp.
New York, N. Y.

### Committee 336, Combined Footings

Fritz Kramrisch
Albert Kahn Associates
Detroit, Mich.

### Committee 337, Strength Evaluation of Existing Concrete Structures

Keith H. Driggs
State of Utah Road Commission
Salt Lake City, Utah

Jacob Feld
Consulting Engineer
New York, N. Y.

James R. Libby
Freyssinet Co., Inc.
East Norwich, N. Y.

Ernst Maag
Department of Public Works
Los Angeles, Calif.

F. W. Panhorst
California Division of Highways
Sacramento, Calif.

Adrian Pauw
University of Missouri
Columbia, Mo.

Abraham Pikoos
E. I. DuPont de Nemours & Co.
Wilmington, Del.

Herbert M. Stoll
A. Epstein & Sons
Chicago, Ill.

### Committee 338, Torsion

David Ehrenpreis
Consulting Engineer
New York, N. Y.

Gordon P. Fisher
Cornell University
Ithaca, N. Y.

Rudolph Szilard
Colorado State University
Fort Collins, Colo.

### Committee 339, Allowable Stresses in Reinforcement

Georg Wästlund
Cement and Concrete Institute
Stockholm, Sweden

### Committee 613, Recommended Practice for Proportioning Concrete

John C. Wycoff
Southern Lightweight Aggregate Corp.
Richmond, Va.

### Committee 623, Cellular Concretes

Irwin A. Benjamin
Granco Steel Products Co.
Granite City, Ill.

George L. Kalousek
Owens-Corning Fiberglas Corp.
Newark, Ohio

Richard J. O'Heir
Perlite Institute
New York, N. Y.

### Committee 717, Practice in Low-Pressure Steam Curing

Robert F. Adams
California Department of Water Resources
Sacramento, Calif.

Carl M. Rollins
Basalt Rock Co.
Napa, Calif.

M. Jack Snyder
Battelle Memorial Institute
Columbus, Ohio

### Committee 805, Application of Mortar by Pneumatic Pressure

Max Dijkmans
The Preload Co., Inc.
New York, N. Y.

Paul J. Fluss
San Francisco Port Authority
San Francisco, Calif.

O. N. Kulberg
Southern California Edison Co.
Los Angeles, Calif.

# Construction Scene

## 1958 Review
## 1959 Outlook

TOTAL CONSTRUCTION performed in the United States, after breaking dollar volume records in the face of the general business decline of 1958, stands a good chance of reaching the $72 billion mark in 1959 with a 6 percent advance, according to The Associated General Contractors of America.

The forecast for 1959 is for about $52 billion in new construction and $20 billion in maintenance and repair; work put in place in 1958 is estimated at nearly $49 billion in new construction and $19 billion in maintenance and repair.

A 16 percent rise in public construction, with an accent on highways, will dominate the 1959 construction scene. In the private sector, a moderate advance in residential activity and continued high levels of commercial and public utility construction should more than offset a further slide in industrial building and a slight drop in farm construction.

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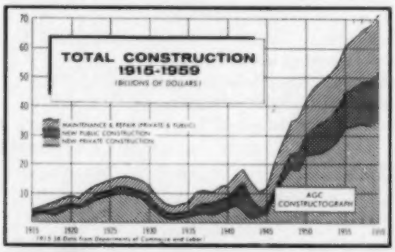
The AGC's annual review and outlook statement estimated that construction of all types in 1958 increased 3 percent over 1957, and saw increasing contract awards and improving economic conditions as the beginning of new construction growth in 1959.

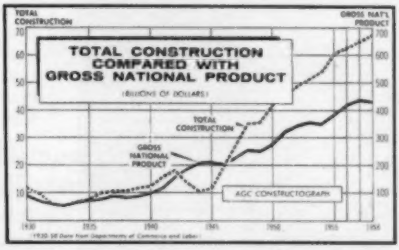
### REVIEW OF 1958 CONSTRUCTION

Last year was the 13th consecutive year in which dollar volume of construction activity, including contract construction and force account operations, increased. A high rate of public works and a surge in residential activity in the closing months pushed new construction to nearly $49 billion, 2 percent above 1957. In terms of physical volume, however, this was offset by an increase in construction costs. These costs, in the aggregate, rose 2.9 percent; wages rose 5.6 percent while basic materials were nearly stable.

For the first time since 1949, private construction, at about $33.9 billion, failed to increase, due primarily to the sharp decline in industrial building resulting from the slump




Construction growth in the United States has been rapid since 1945. In 1958, in face of a general business decline, that growth continued, totaling $67.5 billion




Construction has been taking a larger share of the gross national product since 1945. In 1958 its share was almost 16 percent, shown graphically in chart above

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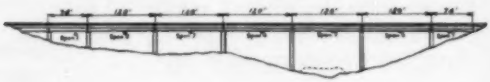

Plastiment concrete in 7 foot deep beams is being revibrated before deck is placed.

This unusual monolithic prestressed beam and slab design resulted from alternate bids permitted by Philadelphia's progressive Department of Streets* . . . Savings of more than 20% over conventional steel design were passed on to the taxpayers.

Each 120 ft. long span, 34 ft. wide; including two traffic lanes, sidewalk, and half of the center median strip was concreted in one continuous 260-yard operation . . . Since 8-10 hours elapsed during placement, Plastiment was specified to control initial set. Plastiment provided these additional benefits: better compaction, greater density, and faster development of stressing strength.

Your clients will benefit from the better structural quality of Plastiment concrete . . . Specify Plastiment for your next job . . . Write for Bulletin PCD.

AD 36-7

*David M. Smallwood, Street Commissioner
Noel W. Willis, Chief Bridge Engineer

The Preload Company, Inc., New York City—Consulting Engineers
The Conduit & Foundation Corporation, Philadelphia—Contractor

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in plant and equipment expenditures which marked the recession. Private industrial building fell more than 30 percent to about $2.4 billion, but was offset by a recovery in residential activity, while the other major business categories held near 1957 levels. Commercial and public utility construction continued at high levels. They were $3.6 billion and $5.6 billion, respectively. Public construction rose 6 percent to $15 billion, sparked by an 8 percent advance in highway construction, which showed the first major effect of the long-range program after the middle of the year. Major public categories were: highways, $5.4 billion; education, $2.9 billion; sewer and water systems, $1.4 billion; military construction, $1.2 billion; conservation and development, $1 billion, and public housing, $830 million.

#### OUTLOOK FOR 1959

The 6 percent increase seen for 1959 will be the largest advance in 4 years, and, if past patterns and long-range forecasts hold true, should signal the start of a new era of construction growth, along with a generally prosperous economy according to AGC forecasters.

The major part of the advance will be in public construction, increasing 16 percent to $17.4 billion, with a significant spur in highways; private construction should increase 3 to 4 percent to nearly $35 billion.

While the AGC new construction estimate approximates that of the government, it differs within the categories. AGC projects lower figures for residential and farm construction and higher estimates for industrial building, highways, public utilities, church, and school construction.

Here is the AGC outlook for major categories:

*Residential*—A 7 percent increase to $19.1 billion in private residential building, and $1 billion in public housing, with upward of 1,190,000 new units started. This rate could be affected by Congressional action on housing and credit proposals. Contract awards indicate a continuing trend of apartment construction toward larger proportions of the residential market.

*Commercial*—Should continue at the peak level of $3.6 billion, with brisk activity in chain stores and shopping centers.

*Religious construction*—May hit the $1 billion mark for the first time in a 15 percent advance.

*Farm construction*—Expected to decline slightly to $1.5 billion.

*Public utilities*—Should hold close to their high levels, and possibly increase slightly to $5.5 billion.

*Industrial building*—Should decline at a reduced rate, about 10 percent, to $2.2 billion. A pickup in capital expenditures throughout 1959 will lay the foundation for a rising volume of business construction in 1960.

*Highways*—Expected to increase more than 15 percent under the impetus of the long-range program, reaching $6.2 billion. A continued advance is seen for 3 or 4 years.

*Public educational building*—May total $3.1 billion, 8 percent over 1958, with 74,000 to 75,000 new public school classrooms constructed.

*Sewer and water facilities*—Will continue to advance, reaching $1.5 billion.

*Military construction*—May advance from $1.2 billion in 1958 to $1.5 billion, with missile bases in the forefront.

*Conservation and development*—Should climb to $1.1 billion, principally from programs administered by the U. S. Bureau of Reclamation and the U. S. Army Corps of Engineers.

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## PCA extends service into northern California and Oregon

The Portland Cement Association recently announced the extension of its field service activities into northern California and Oregon. A suboffice of its Los Angeles district office has been established in San Francisco to service northern California. Cement users in Oregon will be serviced through the Association's Seattle district office.

## Dempsey to head Prestcrete Corporation

John G. Dempsey, formerly general manager of Compania Anonima de Concreto, Venezuela, has assumed the position of president of Prestcrete Corp., Plano, Ill.

Mr. Dempsey is currently a member of ACI Committee 212, Admixtures.

## A. Kleinlogel

A. Kleinlogel died on June 17, 1958, at Darmstadt, Germany. He graduated from the Technical University at Stuttgart in 1900 and worked for many years with Bach and Mörsch.

In 1910 Professor Kleinlogel was awarded a degree of Doctor of Engineering at the Technical University of Dresden for a thesis on the bond between steel and concrete. Shortly thereafter he was appointed a lecturer at the Technical University of Darmstadt, where he also practiced as a consulting engineer. Many of his books and other publications dealt with a wide range of theoretical and practical concrete problems.

In 1922 he became editor of *Beton und Eisen* and remained with the publication until 1943. In 1951 he was honored by the Emil Mörsch medal, the highest distinction in Germany, for achievements in reinforced concrete.

## Wilbur M. Wilson

Wilbur M. Wilson, research professor of structural engineering, Emeritus, University of Illinois, Urbana, died November 28, 1958. A graduate of Iowa State College in mechanical engineering in 1900, he joined the University of Illinois faculty in 1913 and served continuously until his retirement in 1949, except for 2 years 1917-19, when he was an engineering officer in the U. S. Army. He earned a master's degree from Cornell University in 1904 and was awarded the honorary degree of Doctor of Engineering by Iowa State College in 1942.

Professor Wilson's work has included studies of the application of the slope-deflection method to the analysis of statically indeterminate structures, and experimental studies of concrete arches, the bearing value of large rollers, static and fatigue strength of riveted and welded structural joints, and the strength of cylindrical shells. He took an important part in the development of high-strength bolts as a structural fastener, mainly as a replacement for rivets.

Professor Wilson was awarded the Chanute Medal by the Western Society of Engineers in 1915 and 1937; the James J. R. Croes Medal by the American Society of Civil Engineers in 1936; the Wason Medal by the American Concrete Institute in 1939; and the Marston Medal by Iowa State College in 1949.

## Abraham Epstein

Abraham Epstein, president of the firm of A. Epstein and Sons, Inc., Chicago, died Dec. 7, 1958.

Mr. Epstein was born in Kiev, Russia, in 1887 and came to the United States in 1906. He graduated from the University of Illinois in 1911 with a BS in civil engineering.

Prior to engaging in private practice in 1921, Mr. Epstein worked for various architects and the Central Manufacturing District. As consulting engineer to the Central Manufacturing District and the Union Stock Yard and Transit Co., he designed many projects, one of which was the International Amphitheatre.

More recently Mr. Epstein has been responsible for the design of the Borg-Warner Building, Chicago, several buildings for Mount Sinai Hospital, Chicago, and a number of shopping centers, warehouses, and office buildings, outside the Chicago area.

Throughout the years, Mr. Epstein has served on many civic, charitable, and technical committees. At the time of his death he was a member of the Mayor's All-Chicago Citizens Committee and the Committee on Standards and Tests of the Chicago Building Engineers. Mr. Epstein was a long-time member of ACI Committee 318, Standard Building Code. He held membership in a number of technical societies and had been an ACI member since 1930.

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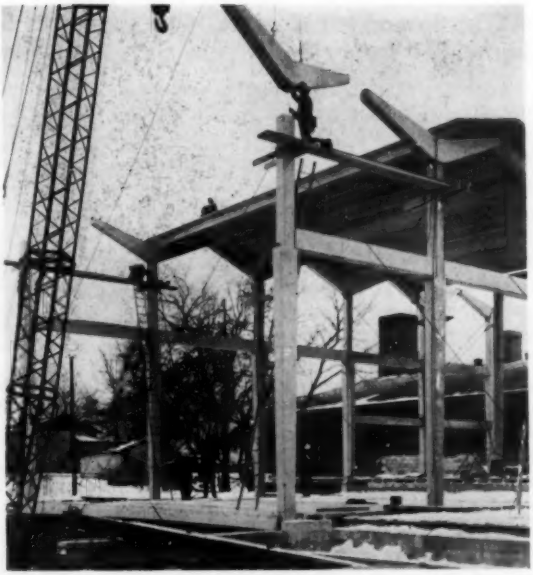
## Permanente Cement promotes Varco

Linc C. Varco has been appointed to the newly-created position of distribution superintendent for the Pacific Northwest division of Permanente Cement Co., it was recently announced by John M. Garoutte, vice-president in charge of operations. Mr. Varco will be responsible for cement distribution from plants at Seattle, Bellingham, Portland, and Pasco, with headquarters in Seattle.

**HIGH ROOF LINE** of St. Michael's Church in Pontiac, Mich., was designed with a multiple gable affect similar to the folded plate construction, using precast concrete



# precast prestressed church roof

The general design of the structure makes use of prestressed double T- and channel slabs for floor and roof skin construction. These units integrally form the joists and deck slabs. The floors were topped with 2 in. of concrete for the finished wearing surface and the roof areas were insulated and membraned.

The slab units for the roof were 49 ft long with 2 ft tapered cantilevers at each end. The ends were tapered from 14 in. in depth at the support to 4 in. at the facia detail. The high roof slabs were supported by shallow V-shaped butterfly beams which were left completely exposed to view. The butterfly beams framed to the tops of tapered columns which in turn were spaced and tied together by spandrel beams at the top ⅓ point. The arrangement and combination of the butterfly and spandrel beams bordered by the columns form an esthetic frame for the 2½ in. thin precast concrete window frames. The precast concrete window frames were freely used for both interior and exterior architectural concrete work.

The structure was designed by Charles D. Hannan; Richard McHugh Co. served as general contractor; prestressed units manufactured and erected by American Prestressed Concrete, Inc.

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## Gray becomes partner in firm of DeLeuw, Cather and Brill

Benjamin Gray has become a general partner of DeLeuw, Cather and Brill, New York City architects and engineers, according to a recent announcement made by the company's managing partner, C. B. F. Brill.

Mr. Gray was formerly chief architect for the engineering firm of Burns and Roe, Inc.

DeLeuw, Cather and Brill is affiliated with DeLeuw, Cather and Co. of Chicago, San Francisco, and Toronto.

## Gillis and Forell form partnership

William M. Gillis, architect, and Nicholas F. Forell, structural engineer, have announced the formation of a partnership and have established an office in San Francisco.

Mr. Gillis, graduate of the University of California, was formerly a partner in the firm of John Lyon Reid, architects and engineers, San Francisco; Mr. Forell, graduate of Brown University, Providence, R. I., was formerly senior engineer with the same organization.

## ASTM Committee Week to be held February 2-6

Approximately 1500 to 2000 members, committee members, and visitors are expected to attend the American Society for Testing Materials Committee Week meeting in Pittsburgh at the Penn-Sheraton Hotel, Feb. 2-6.

About 30 of the Society's main technical committees will hold some 350 meetings of subcommittees and working groups to conclude the year's committee work to enable preparation of reports for presentation at the annual meeting at Atlantic City in June.

John C. Warner, president, Carnegie Institute of Technology and Technical Consultant to the U. S. delegation to the Second U. N. International Conference on the Peaceful Uses of Atomic Energy at Geneva, Switzerland, will be the guest of honor and speaker at the Committee Week dinner on February 4.

## Cordova moves from Westville to Gary

The firm of Francisco J. Cordova, engineers and geologists, has moved its engineering offices, testing laboratory, and personnel from Westville, Ind., to new quarters in Gary.

## Ayres appointed vice-president of Haller Testing Laboratories

Elliot A. Haller, president of the Haller Testing Laboratories, Inc. with headquarters in New York City, has announced the appointment of George H. Ayres as vice-president.

Mr. Haller has been with the firm since 1953 and has been manager of the New York division since 1954, in charge of sales, supervision of test reports, and operations in that area. He assumes similar duties at the New Haven, Conn., division of the company.

## Myer joins sales staff of Southern Lightweight Aggregate

Thomas L. Myer is the new sales representative in charge of Southern Lightweight Aggregate Corporation's office for the Washington, D. C., area, located in Bethesda, Md. Mr. Myer has been active in the construction and supply business in western North Carolina for the past 13 years.

The Southern Lightweight Aggregate Corp. maintains plants in Bremo Bluff and Leaksville Junction, Va., and in Aquadale, N. C. A new million dollar plant, now under construction, will go into operation in Green Cove Springs, Fla., in March, 1959.

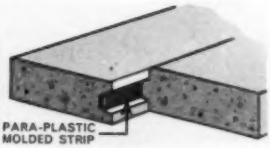
**UNIQUE THIN SHELL CONCRETE VAULTS** cover the main building areas at all of the Santa Monica Beach Parks. The small barrels are 2½ in. thick and were placed three at a time. The shells are ribless and the thrust at the columns is taken by bending of the end columns. The interior shells are considered to balance one another for thrusts. Richard R. Bradshaw, Los Angeles, served as structural engineer on the buildings.

# SERVICISED PREMOLDED RUBBERIZED ASPHALT SEALER STRIPS

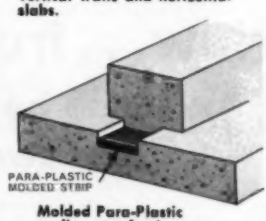
## MOLDED *Para-Plastic*®

Molded Para-Plastic is a durable, elastic compound, composed of rubber, asphalt and other materials, molded into a rectangular strip. Properly installed, Molded Para-Plastic will bond securely to concrete to form a resilient and moisture-tight seal of the joint.

Because it presents sealing surfaces on all sides, Molded Para-Plastic is especially effective in sealing keyed construction joints and foundation footings. Not recommended for use in joints where the surface of the material is exposed, or when hydrostatic pressure is present. Available in various sizes with thickness ranging from ¼" to 1" and widths from 1 to 12 inches. Standard length is 5 feet.



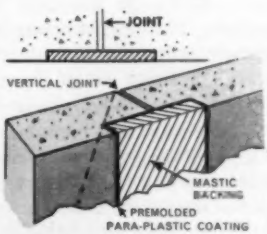

**Keyed construction joint in vertical walls and horizontal slabs.**




**Molded Para-Plastic sealing at a footing.**

## PARA-LATERAL STRIP

Consists of a rigid mastic backing material coated on one side and two edges with a Para-Plastic rubber-bituminous sealing compound which bonds firmly to the concrete after it has set up, to form a watertight seal. Designed for use in vertical and sloping joints in concrete retaining walls, abutments, wing walls, foundation walls, tunnels, etc., where backfilling against one side of the structure requires a seal to prevent seepage of ground water through the joint to the exposed side. Available in widths of 4", 5", 8", 12", and 13"; standard length is 5 feet.




**Application of Para-Lateral Sealing Strip to rear face of wall at joint.**

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**PLAIN**
**1 : 2 : 4 Mix**

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**WITH PLACEWEL**
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*You get all the advantages of entrained air without its adverse effects on strength.*

Less than 3 oz. Placewel/sack portland cement will—

- increase strength 30% or more (after 28 days' curing)
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# Honor Roll

## January 1—December 31, 1958

This Honor Roll lists 464 "member conscious" ACI'ers who have added more than 1350 new members to the Institute. Each of these proposers has enabled ACI to do more for all its 9686 members and, we believe, has done a favor for each of the new members.

Blas Lamberti....15
Joseph J. Waddell....14
Phil M. Ferguson....11
James A. McCarthy....11
C. P. Siess.... 9
L. M. Legatski.... 8
Robert P. Witt.... 8
Douglas McHenry.... 7
Martin J. Gutzwiller.... 6
Ernst Maag.... 6
H. C. Pfannkuche.... 6
Jerome M. Raphael.... 6
J. P. Thompson.... 6
M. A. Craven.... 5
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P. E. Ellen.... 5
A. T. Hersey.... 5
Samuel Hobbs.... 5
Carl H. Koontz.... 5
Ernest L. Spencer.... 5
Clarence A. Walkwitz.... 5
Moises Bendahan.... 4
Celso A. Carbonell.... 4
Vincent R. Cartelli.... 4
Jaime de las Casas.... 4
N. J. Everard.... 4
E. L. Howard.... 4
Frank E. Legg, Jr.... 4
Daniel S. Ling.... 4
George A. Mansfield.... 4
John B. Porter.... 4
Walter H. Price.... 4
E. Schmid.... 4
C. H. Scholer.... 4
Howard Simpson.... 4
Otis D. Small.... 4
Luther E. Bell.... 3
Aleck S. Evans.... 3
Louis A. Gottheil.... 3
Richard W. Gunn.... 3
Paul A. Hansen.... 3
Robert B. Harris.... 3
Kenneth M. Huber.... 3
Thomas C. Kavanagh.... 3
Joe W. Kelly.... 3
E. Vernon Konkel.... 3
E. Richard Kummerle.... 3
Charles Macklin.... 3
Bryant Mather.... 3
H. P. Mittet.... 3
Jorge L. Molina M.... 3
Everette W. Osgood.... 3
Francesco Sardella.... 3
Joseph J. Shideler.... 3
O. C. Struthers.... 3
T. W. Thomas.... 3
Ruben Cano Vicario.... 3
Ellis S. Vieser.... 3
Byron P. Weintz.... 3
William M. Avery.... 2
Robert B. Banning.... 2
Nicandro Jose Barboza.... 2
Ira M. Beattie.... 2
O. R. Bell.... 2
Massimiliano R. Berretti.... 2
Carlos D. Bullock.... 2
S. D. Burks.... 2
Antonio Cajigas.... 2
Miles N. Clair.... 2
W. L. Collier.... 2
W. S. Cottingham.... 2
Richard M. Dillon.... 2
Lancelot A. Fekete.... 2
Enrique Garcia-Reyes.... 2
Thor Germundsson.... 2
Myron L. Goral.... 2
Einar Block Hansen.... 2
Arch Hull.... 2
Jorge Herrera Ibarra.... 2
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O. G. Julian.... 2
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R. G. King.... 2
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Ciceron Hiedra Lopez.... 2
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I. E. Morris.... 2
James A. Murray.... 2
D. F. Nash.... 2
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George H. Nelson.... 2
Tatuo Nisigori.... 2
Ramiro Parada.... 2
Neftali Penaloza R.... 2
J. L. Peterson.... 2
Victor M. Rada A.... 2
Abdur Rahman S. Rasul.... 2
Clarence Rawhauser.... 2
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Herbert Rusk.... 2
Sabri Sami.... 2
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Gordan H. Chapman ..... 1
Mauricio Chedraui ..... 1
Lawrence D. Childs ..... 1
James Chinn ..... 1
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Edward Cohen ..... 1
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Walter E. Comeadow ..... 1
Harlan Coornvelt ..... 1
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William V. Coyle ..... 1
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Ernest A. Dukleth ..... 1
H. J. Dunasky ..... 1
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Ward W. Engle ..... 1
Dogan H. Erokan ..... 1
Frank G. Erskine ..... 1
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Andrew G. Fallat ..... 1
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## Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. The grades of membership are described overleaf.

All who have an interest in concrete are eligible for membership.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI JOURNAL provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

Opportunity for service is present in technical committee activity, in contributions, or only comments, to the ACI JOURNAL, or in reviewing technical publications for material of interest to the membership.

ACI's world-wide membership is growing in extent and participation—traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

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(cut here)

**Board of Direction, American Concrete Institute** Date_____
**P. O. Box 4754, Redford Station**
**Detroit 19, Michigan**

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|---|---|
| Individual Members (U. S. and Possessions, Canada, Mexico, Central America, and West Indies) | $15.00 |
| Individual Members (All other foreign countries) | 12.00 |
| Corporation Members | 50.00 |
| Contributing Members | 100.00 |
| Junior Members—nonvoting (under 28) | 7.50 |
| Student Members—nonvoting (under 28) | 5.00 |

(Subject to stipulations of Bylaws—Article I on reverse side. Bylaws on request.)

Of the annual dues, $12.00 is for the JOURNAL of the American Concrete Institute (except that dues for Junior and Student Members apply in full for the JOURNAL).

The undersigned hereby applies for_____membership
(Individual, Corporation, Contributing, Junior, Student)

in the American Concrete Institute. Proposed by_____

For Corporation Membership, ACI representative will be_____

_____ _____
(Date of graduation if Student) (Name, if Corporation)

Signature_____

For our records, please complete both sides of application.

## EXCERPTS FROM BYLAWS: ARTICLE I—MEMBERS

*Section 1.* This Institute shall consist of Honorary Members, Corporation Members, Contributing Members, Members, Junior Members and Student Members.

*Sec. 3.* A Member shall be a person.

A Corporation Member shall be a firm, corporation, society, agency of government, or other organization.

A Contributing Member shall be a person, firm, corporation, society, agency or government, or other organization electing to give greater support to Institute activities through the payment of larger dues. Any Contributing or Corporation Member, other than a person, may name a personal representative who shall enjoy all membership rights and privileges.

A Junior Member shall be a person less than 28 years of age.

A Student Member shall be a person less than 28 years of age and a registered student at a technical or engineering school.

*Sec. 4.* All classes of Members, except Honorary Members and Student Members, shall be sponsored by at least one Member of the Institute. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be sponsored either by a Member of the Institute or by a member of his school's faculty, who need not be a Member of the Institute.

*Sec. 5.* All Members in any classification shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall neither vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member or Member, depending on age, on the first anniversay of his membership succeeding the date on which he ceases to be a registered student. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

*Sec. 6.* Applications for and resignations from membership and requests for change of representatives of Corporation or Contributing Members shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

---

(cut here)

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Please Print

**Date of Birth** _____
Month Day Year

**Title or Position** _____

**Name of Firm or Organization** _____

☐ **Business Address** _____

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(Please check address to which you wish mail and publications sent)

**Nature of Firm's Business** _____

**The ACI Membership Directory will be sent—as available—only on request. Check here if you wish to receive the latest edition.** ☐

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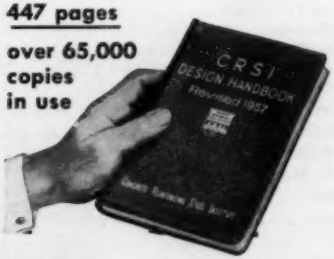


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# New Members

**The Board of Direction approved 72 Individual applications, 3 Corporation, 19 Junior and 20 Student applications, making a total of 114 new members. Considering losses due to resignations, deaths, and nonpayment of dues the total membership on December 1, 1958 was 9693.**

### Individual

ADAMS, ROLAND E., Milwaukee, Wis. (Engr., Supv., Klug & Smith Co.)
ALLEN, DR. I., JR., Bound Brook, N. J. (Sr. Res. Assoc., Bakelite Co.)
BAVE, GUNNAR, Stockholm, Sweden (C. E., Internationel la Siporex AB)
BENVENISTE, JACQUES E., New York, N. Y. (Lecturer, The City College, Dept. of C. E.)
BICKERS, MICHAEL S., Mercury, Nev. (Asst. Proj. Engr., Holmes & Narver, Inc.)
BLETZACKER, RICHARD W., Grove City, Ohio (Supv. Bldg., Research Staff, Engineering Experiment Station, Ohio State Univ.)
BOWEN, DONALD E., Pontiac, Mich. (Partner, Construction Chemicals Co.)
BRADSTOCK, JOHN W., Toronto, Canada (Reicher, Bradstock & Assocs., Ltd., Cons. Engrs.)
BRITTON, HAROLD, Albany, N. Y. (Sr. Struct. Spec. Writer, New York State Dept. of Pub. Wks.)
BRODEUR, J. C., Montreal, Canada (Vice-Pres. & Gen. Mgr., Spencer, White & Prentis of Canada, Ltd.)
BURGESS, WOODROW L., Cleveland Heights, Ohio (Asst. Dir. of Engrg., The Master Builders Co.)
BURKE, W. F., Burlington, Mass. (Asst. C. E., Commonwealth of Mass., Metropolitan Dist. Comm. Constr. Div.)
CARRUTHERS, C. D., Toronto, Canada (Pres., C. D. Carruthers & Wallace, Consultants, Ltd.)
CORNWELL, JOHN E., West Chester, Pa. (Retired farmer)
CORSELL, PETER P., Oak Park, Mich. (Dist. Mgr., L. Sonneborn Sons, Inc.)
DAWSON, ARTHUR R., Detroit, Mich. (Sr. Constr. Insp., City of Detroit, D. P. W., Bur. of Test & Insp.)
DI FIORE, JOSEPH, New York, N. Y. (Cons. Engr.)
DOEDE, ORVIN T., Rosholt, Wis. (Constr. Supt. & Est., Doede Constr. Co.)
DURKIN, DESMOND, Liverpool, England (Civil & Struct. Engr., Huckle & Durkin)
ENGSTRAND, BEN GUNNER, JR., Pasadena, Calif. (C. E., Struct. Designer, Sandberg-Serrell)
FISCHER, ROBERT E., New York, N. Y. (Assoc. Editor, Engrg., Architectural Record, F. W. Dodge Corp.)
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Feb. 2-6, 1959—American Society for Testing Materials, Committee Week, William Penn Hotel, Pittsburgh, Pa.

Feb. 15-19, 1959—National Sand and Gravel Association, 43rd Annual Convention, Hotel Roosevelt, New Orleans, La.

**Feb. 23-26, 1959—American Concrete Institute, 55th Annual Convention, Statler Hilton Hotel, Los Angeles, Calif.**

Apr. 6-8, 1959—8th Annual Meeting, Building Research Institute, Penn-Sheraton Hotel, Pittsburgh, Pa.

May 13-15, 1959—"Deutsche Betontog 1959," German Concrete Association, Congress Hall of the German Museum, Munich, Germany

May 18-19, 1959—39th Annual Meeting, The Society of American Military Engineers, Washington, D. C.

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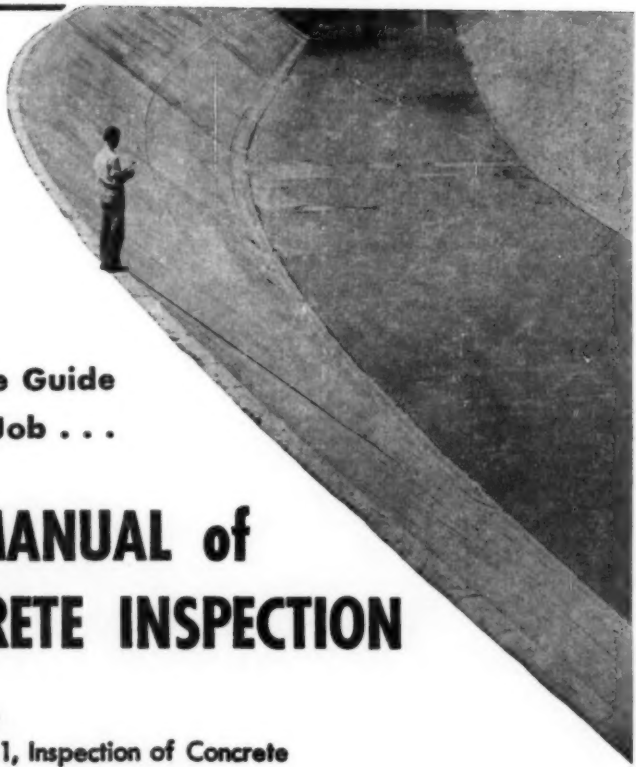

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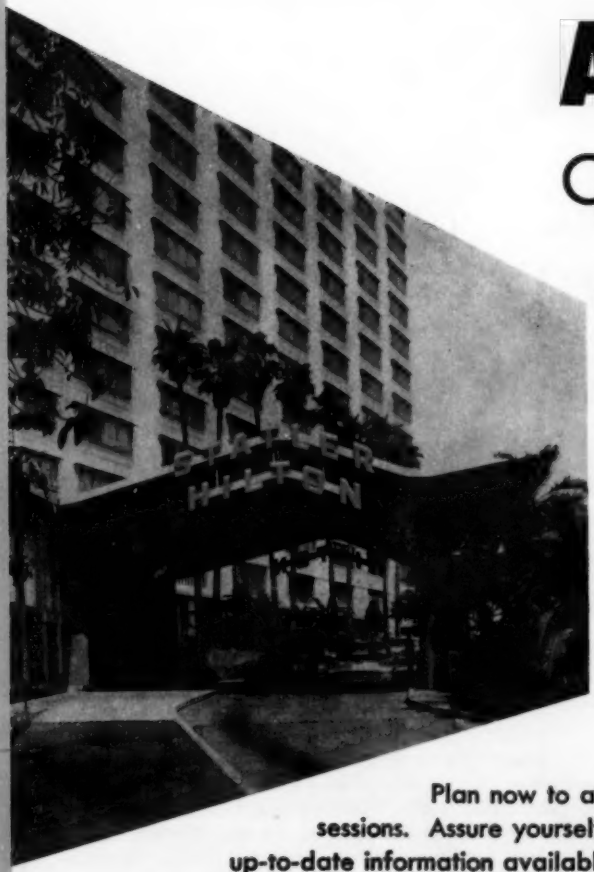


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